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City of Regina

Intermodal Facility and Industrial Lands Servicing Study

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1. Introduction

The City is proposing to annex approximately five (5) sections of land within the Rural Municipality of Sherwood to plan for future industrial growth that extends beyond the current western City limits. Drawing 01 – Annexation Areas Servicing Review included in the back of this report illustrates the limits of the annexation area in relation to the existing City limits and the Airport. Note that there are a series of common drawings that have been grouped in the back of the report in Appendix A (Drawing 1-11). These drawings were grouped together for quick reference as they are referred to at different points throughout the document. All other figures and tables are presented with the body of the report.

The new industrial development is consistent with recommendations made in the recently completed Industrial Growth Study[1] conducted by MKI, as well as the Intermodal Facility (IMF) requirements. In order to plan for the annexation of this land, a concept plan and municipal serviceability analysis needs to be determined.

The scope of work included the identification of overall servicing concepts for road, water, wastewater and storm water services, as well as analysis of the municipal servicing financial implications associated with annexation and development of the area. The serviceability analysis also provided a comparative review of predicted system demands (e.g. peak and long term water supply requirements and wastewater generation quantities) to existing system capacities.

Demands and capacities were based on skeletal network modeling and available system/model information, as provided by the City. The overall objective of the study was to provide the City with a relative level of confidence that the connection(s) to the existing City systems (i.e. water supply and effluent discharge points) were technically feasible and could be engineered to substantiate development within the annexation area, based on the information provided by the City.

[1] *City of Regina. (August 2007). "Industrial Growth Study – Final Report".*

2. Land Use Plan - Phasing & Staging

2.1 Conceptual Land Use Plan

The development area will consist of two major land use categories, i.e., Dry Industrial and Mixed Industrial as presented in Drawing 02 - Land Uses – Servicing Review. The IMF and Loblaw's facility as well as the surrounding parcels contained within the annexation are assumed to be warehousing (light) industrial land use types, i.e., Dry Industrial. The areas east of Condie Road are assumed to be consistent with medium industrial land use types, i.e., Mixed Industrial. There will be no heavy Industrial development expected in the area according to the planning study that has been conducted as part of the annexation review.

The study area includes the following major sections of land:

- Section 28 - Township.17 - Range. 20 - W2M, including Dewdney Avenue
- Section 21 - Township.17 - Range. 20 - W2M, all land north of CPR Mainline,
- Section 20 - Township.17 - Range. 20 - W2M, all land north of CPR Mainline.
- Section 19 - Township.17 - Range. 20 - W2M, including Dewdney Avenue
- Section 24 - Township.17 - Range. 21 - W2M, including Dewdney Avenue
- Northwest 1/4 Section 17 - Township.17 - Range. 20- W2M, all land north of CPR Mainline.
- North 1/2 Section 18 - Township.17 - Range. 20 - W2M, all land north of CPR Mainline.
- North 1/2 Section 13 - Township.17 - Range. 21 - W2M, all land north of CPR Mainline.

2.2 Phasing and Staging

The proposed phasing and staging presented in Drawing 03 – Phasing and Staging – Servicing Review indicates that the Industrial areas annexed to the west of the City of Regina will develop in two phases. Phase I is expected to develop first and has been disaggregated into three separate stages.

2.2.1 Phase I (Short, Medium and Long Term Development)

Phase I includes Sections 20, 19, 24 and portions of Section 13, 18 and 17, which are located above the CPR Mainline right-of-way. This phase will be staged in three different time periods as summarized in Table 1 – Land Use Schedule.

The description and location of each stage follows.

2.2.1.1 Stage I - Short Term Development

Stage I represents the Short Term Development stage which is expected to occur within the next 3 years. Stage I will consist of Dry Industrial land uses encompassing approximately 194 Ha of area. The IMF which is located north of the CPR Mainline in the north half of Section 13 and northwest ¼ of Section 18 and the Loblaw warehouse which is located in the southwest ¼ of Section 19 will be part of this stage.

2.2.1.2 Stage II - Medium Term Development

Stage II represents the Medium Term Development stage which is expected to take place within 3 to 25 years. Stage II will consist of Dry and Mixed Industrial land uses encompassing an additional 459 Ha of area. The land for this stage includes south half of ½ Section 24, southeast ¼ of Section 19, northeast ¼ of Section 18 north of the CPR Mainline, northwest ¼ of Section 17 and the entire Section 20.

2.2.1.3 Stage III - Long Term Development

Stage III represents the Long Term Development stage which will be developed beyond the 25-year planning horizon. It will consist of Dry Industrial land uses and encompass a gross area of 256 Ha. The land for this stage includes north ½ of Section 24 and north ½ of Section 19.

2.2.2 Phase II - Long Term Development

Phase II is being considered as Long Term Development beyond 25 years and will consist of Mixed Industrial land uses. The gross area of this phase is 365 Ha. Phase II development includes lands within Section 21 north of CPR mainline and Section 28.

Table 1 – Land Use Schedule presents a summary of the land uses and the expected scheduling of the various phases and stages described above. It also shows the expected gross land area and the actual land area that will be developed within each separate stage.

Table 1 – Land Use Schedule

PHASE I			
Node	Land Use	Gross Area (Ha)	Developed Area (Ha)
Phase I - Stage I (Short Term 0 - 3 years)			
Intermodal Facility (CPR)			
N Section 13 (North of CPR Mainline)	Industrial	79	4.0
NW Section 18 (North of CPR Mainline)	Industrial	51	2.5
Loblaw			
SW Section 19	Dry Industrial	64	48.0
Subtotal Stage I		194	54.5
Phase I - Stage II (Medium Term 3 - 25 years)			
Dry Industry Area West Pinkie Hwy			
SE Section 24	Dry Industrial	64	48.0
SW Section 24	Dry Industrial	64	48.0
NE Section 18 (North of CPR Mainline)	Dry Industrial	17	12.5
SE Section 19	Dry Industrial	64	48.0
Mixed Industrial Area			
NW Section 20	Mixed Industrial	64	48.0
SW Section 20	Mixed Industrial	64	48.0
NW Section 17 (North of CPR Mainline)	Mixed Industrial	7	5.3
E Section 20 - Sakimay	Mixed Industrial	115	86.3
Subtotal Stage II		459	344.1
Phase I - Stage III (Long Term beyond 25 years, above 235,000 population)			
Dry Industry Area West Pinkie Hwy			
NW Section 19	Dry Industrial	64	48.0
NE Section 19	Dry Industrial	64	48.0
NW Section 24	Dry Industrial	64	48.0
NE Section 24	Dry Industrial	64	48.0
Subtotal Stage III		256	192.0
Total Phase I		909	590.6
PHASE II			
Node	Land Use	Gross Area (Ha)	Developed Area (Ha)
Mixed Industrial Area East Pinkie Hwy (Long Term beyond 25 years, above 235,000 population)			
Section 28	Mixed Industrial	165	123.8
Section 21	Mixed Industrial	200	150.0
Total Stage I		365	273.8
Gross Area Land		1274	864.4

2.3 Equivalent Population Estimates

The City of Regina Development Standards Manual 2008 – Section 10 Table 4.5.3 suggests using a density of 25 persons/Ha for Industrial Land Use. This standard generates flows that are consistent with light industrial or dry warehouse style industrial land use based on design criteria found in other jurisdictions. Design criteria in other jurisdictions also suggest doubling the flows for mixed industrial land use relative to light industrial land use. This however assumes that some heavy industrial land use will be included in the mix. Since this is not the case for the lands considered within the annexation boundary we have assumed an increase of roughly 50% or 35 persons/Ha for mixed industrial land use.

Table 2 – Land Use Design Population Densities is a summary of the densities that were assumed in the study.

Table 2 – Land Use Design Population Densities

Land Use	Design Population (persons/Ha)
Commercial	65
Industrial	25
Mixed Industrial	35
Mixed Use	80
Residential	40

3. Transportation

The City is proposing to annex just over five (5) sections of land within the Rural Municipality of Sherwood to plan for future industrial growth that extends beyond the current westerly City limits. In assisting with the development of the concept plan, this review identifies the overall roadway servicing concept required to support the developed area.

The goal of the transportation review is to re-confirm road network deficiencies and to identify any additional network upgrades required to support the fully developed IMF industrial growth area and potential surrounding RM development. Road network modifications already recommended through other studies, namely the Industrial Growth Study, were considered within the analysis. Improvements examined included a combination of geometric, traffic control or structural improvements. The analysis examines the effect of a combination of new roadway links and upgrades to existing roadways.

The amount of traffic generated by the IMF area for the forecast weekday afternoon peak hour was established. The distribution and assignment of new traffic trips to the road network will determine future road infrastructure requirements. The forecast timeframe for analysis corresponds to traffic volumes and road network developed for a 235,000 population forecast scenario, typically considered to be a 20-25 year timeframe.

3.1 Previously Identified Road Network Upgrades

The following key roadway infrastructure upgrades required to service the new industrial growth area as well as other key new roadways within the southwest have previously been identified in the City of Regina Road Network Plan, as well as in the Southwest Sector Road Network Study:

- Four new interchanges have been identified:
  - an interchange at the intersection of Lewvan Drive and Highway No. 1 (the detailed design is currently in progress and construction is anticipated to occur within the next several years)
  - an interchange at the intersection of Pinkie Road and Highway No. 1
  - an interchange at the intersection of Pinkie Road and Highway No. 11
  - an interchange at the Saskatchewan Drive and Lewvan Drive intersection
- The reconstruction of Pinkie Road (i.e. the West Bypass) from Highway No. 1 to Highway No. 11. Pinkie Road is currently a 2-lane gravel road, with a paved section between the CPR tracks and Dewdney Avenue. Pinkie Road was identified as the westerly corridor for the National Highway Network in the 1996 Regina Regional Highway Plan, as well as the western part of the City of Regina bypass. The route location study for Pinkie Road is currently being undertaken
- Portions of Dewdney Avenue west of Lewvan Drive have also been identified as requiring structural upgrades to accommodate an increase in heavy truck traffic. Dewdney Avenue is currently a heavy truck route through the City of Regina
- The extension of Courtney Street south, as well as the extensions of Gordon Road and Parliament Avenue west to connect with Courtney Street

3.2 Development of New Vehicle Trips

The trip estimates include both vehicle and truck traffic generated from the following two areas:

- The lands within the annexation boundary: This land includes the IMF, Loblaws and three other Logistics “Superparks” and the lands set aside for their expansion, and additional industrial lands for conventional industrial development
- Section 16 immediately outside of the annexation boundary: The north half of this section is located outside of the annexation boundary, but has been identified as having potential for industrial development. It was determined that any trips resulting from this development should be included in the traffic analysis, as they would have an impact on the capacity and operations of Dewdney Avenue and Pinkie Road

Vehicle and truck trips are based on the number of employees forecast for each individual land use within the concept plan. MKI provided estimates of employment totals for the concept plan. Where available, actual employment totals and/or trip information as provided by CPR and Loblaw’s for their sites was used. Loblaw’s provided both the number of hourly truck trips as well as the number of hourly employee trips generated by the Loblaw’s site.

The yearly truck trips generated by the IMF, as well as the number of employees forecast to work at the IMF, were also provided by CPR. It is estimated that a total of 150 employees are forecast to work at the IMF. The following assumptions were used to develop hourly truck trips for the IMF:

- In 2007, the existing IMF was handling up to 40,000 container trips per year
- A multi-year projection shows a growth of 7 percent annually with the terminal reaching a total of 217,000 container trips per year by the forecast year of 2031 (600 trucks per day)
- A more aggressive forecast (20 percent growth annually for the first 5 years, then a 7 percent growth for the remaining 15 years) projects a total of 385,000 container trips per year (1050 trucks per day)

The forecast truck trips per hour generated by the IMF was developed by assuming that 7 percent of the daily truck trips would occur during the afternoon peak hour. This represents an average of 60 truck trips per hour.

Trips for the industrial land uses were then calculated using trip generation rates derived from the Institute of Transportation Engineers (ITE) - 7th Edition Trip Generation Manual. Logarithmic equations, where available, were used in place of average rates. The number of new trips generated is based on several assumptions. Trips were based on the number of employees forecast for the industrial park (medium) and warehousing (light) industrial land use types. The IMF and the Logistics Superparks were assumed to be warehousing type land uses. The remaining industrial developments were assumed to be light industrial land uses.

The total number of new vehicle trips generated for the industrial growth area for the weekday afternoon peak hour is shown below in Table 3 – Afternoon Peak Hour Trip Generation.

A total of 13,695 employees are forecast for this area. The total number of trips generated based on the employment forecast is estimated at 5,685 trips (1,460 trips in and 4,225 trips out) during the weekday afternoon peak hour. Truck percentages are estimated at 15 percent, corresponding to a total of 855 trucks per hour during the afternoon peak hour. The average trip rate is 0.5 trips per employee. The distribution of trips is on average 25 percent inbound and 75 percent outbound during this time period.

Three staging scenarios will be reviewed:

- **Phase 1** – A 3-year forecast. Immediate known development within the annexation lands includes the IMF and Loblaws site.
- **Phase 2** – A 25-year horizon. Includes all remaining lands west of the current Pinkie Road alignment (with the exception of the north half of Section 19 and the north half of Section 24).
- **Phase 3** – beyond a 25-year horizon. This land includes all long term development located within the annexation lands, including the north half of Section 19 and the north half of Section 24, and Sections 21 and 28.

Table 3 – Afternoon Peak Hour Trip Generation

Industrial Park Land Use Types	Employees	Trip Rate	Trips Gen.	% In	% Out	Trips In	Trips Out	Total Trips
				Afternoon Peak Hour Traffic Volumes				
Phase 1 - Immediate Known Development								
IMF Facility - Employee Trips	150	0.6	95	35%	65%	35	60	**95**
IMF Facility - Truck Trips	-	-	60	35%	65%	20	40	**60**
Logistics Superpark - Phase 1 (Loblaws)	1,200	0.3	315	54%	46%	170	145	**315**
Logistics Superpark - Phase 1 (Loblaws) -	-	-	20	50%	50%	10	10	**20**
	1,350	**0.4**	**490**		**Total Trips**	**235**	**255**	**490**
					Truck Trips	**30**	**50**	**80**
					Vehicle Trips	**205**	**205**	**410**
Phase 2 - up to 25 Years (to 2031)								
Logistics Superpark - SE Section 24	284	0.7	185	35%	65%	65	120	**185**
Logistics Superpark - SW Section 24	256	0.7	170	35%	65%	60	110	**170**
Logistics Superpark - SE Section 19	383	0.6	225	35%	65%	80	145	**225**
Industrial Park - W1/2 Section 20	1,679	0.4	680	20%	80%	135	545	**680**
Industrial Park - E1/2 Section 20	1,710	0.4	690	20%	80%	140	550	**690**
	4,311	**0.5**	**1,950**		**Total Trips**	**480**	**1,470**	**1,950**
					Truck Trips	**70**	**220**	**295**
					Vehicle Trips	**410**	**1,250**	**1,655**
Phase 3 - Long Term Industrial (beyond 2031)								
Industrial Park - N1/2 Section 19	567	0.5	310	35%	65%	110	200	**310**
Industrial Park - N1/2 Section 24	567	0.5	310	35%	65%	110	200	**310**
Industrial Park - W1/2 Section 28	1,920	0.4	755	20%	80%	150	605	**755**
Industrial Park - Section 21	2,880	0.4	1,055	20%	80%	210	845	**1,055**
Industrial Park - N1/2 Section 16	2,100	0.4	815	20%	80%	165	650	**815**
	8,034	**0.4**	**2,625**		**Total Trips**	**745**	**2,500**	**3,245**
					Truck Trips	**110**	**375**	**485**
	13,695				**Vehicle Trips**	**635**	**2,125**	**2,760**

Total Trips - Up to 2031	**715**	**1,725**	**2,440**
Total Trips - Long Term Beyond 2031	**745**	**2,500**	**3,245**
Total Trips	**1,460**	**4,225**	**5,685**

3.3 Trip Distribution

The direction or trip distribution from which traffic will approach and depart the IMF study area can vary depending on several factors, most importantly, the type of trip (truck versus non-truck) and the distribution of population and employment and characteristics of the surrounding road network.

The distribution presented in Table 4 – Hourly Truck Trip Distribution was used to distribute the truck trips to and from the road network. It is based on the following:

- The truck trip distribution is based on 2007 daily traffic volumes and commercial percentages on major highways in and out of the Regina area (i.e. north - Highways No. 11, 6; south – Highway No. 6; east – Highways No. 46, 1 and 33; and west – Highway No. 1)
- Internal trips are intermodal truck trips that originate or terminate at a location within Regina. These trips would primarily use existing City of Regina truck routes such as Dewdney Avenue to make trips to and from the new IMF.

Table 4 – Hourly Truck Trip Distribution

Direction	Percent	Phase 1 Truck Totals	Phase 2 Truck Totals	Phase 3 Truck Totals	Total
Internal	10%	10	30	50	90
North	25%	20	75	120	215
South	10%	10	30	50	90
East	35%	25	100	170	295
West	20%	15	60	95	170
Total	100%	80	295	485	860

The trip distribution for vehicle trips is based on population forecasts used in the City of Regina's EMME/2 model developed for the Northwest, Southeast and Southwest road network studies. The distribution shown in Table 5 – Hourly Vehicle Trip Distribution was used to distribute the vehicle trips to and from the road network.

Table 5 – Hourly Vehicle Trip Distribution

Direction	Percent	Phase 1 Vehicle Totals	Phase 2 Vehicle Totals	Phase 3 Vehicle Totals	Total
External	5%	20	85	140	245
North	20%	80	330	550	960
South	30%	125	495	830	1,450
East	30%	125	495	830	1,450
West	15%	60	250	410	720
Total	**100%**	**410**	**1,655**	**2,760**	**4,825**

3.4 Trip Assignment

The final step to developing traffic volumes is traffic assignment - the process of routing site-generated trips to and from the site onto the surrounding road network. Traffic assignments should consider logical routings, available, current and projected roadway capacities, and travel times. The key assumption in this trip assignment for this analysis was that the approved City of Regina Road Network Plan improvements were in place, as well as updates recommended through the City of Regina's *Southwest Sector Road Network Study, Southeast Sector Road Network Study* and *Northwest Sector Road Network Study.*

The truck trip assignment for the study area considered the following assumptions:

- Direct access to Pinkie Road was not assumed – access will be provided via service roads
- Internal trips would primarily use existing City of Regina truck routes such as Dewdney Avenue and Lewvan Drive to make trips to and from the new IMF. A small percentage of these internal trips may choose to use Pinkie Road to access Highway No.1 and onto other destinations within Regina
- Traffic to and from the north (i.e. Highway No. 6 North or Highway No. 11) will have several options to access the area – an internal route using City of Regina truck routes (Dewdney Avenue to Lewvan Drive/Pasqua Street or Albert Street), or an external route accessing Pinkie Road from Highway No. 11 (and bypassing all city of Regina roadways) when the interchange at Highway No. 11 and Pinkie Road is upgraded
- Traffic to and from the east and south (i.e. Highway No. 46, Highway No. 1, Highway No. 33 and Highway No. 6 South) will have two routing options to access the intermodal site – the use of Pinkie Road or the use of the Lewvan Drive and Dewdney Avenue corridors. It was assumed that the preferred route would be Pinkie Road
- Traffic to and from the west (i.e. Highway No. 1 West) will also have two options to access the intermodal site – the Lewvan Drive/Dewdney Avenue corridors or the Pinkie Road corridor. It was assumed that the majority of trucks and vehicular traffic would use the Pinkie Road corridor

Vehicle trip assignment was distributed evenly between Dewdney Avenue and Pinkie Road south of Dewdney Avenue. It was assumed that traffic assigned to Dewdney Avenue would eventually utilize Courtney Street and Lewvan Drive to access the north, south and west sections of Regina.

3.5 Background Traffic Volumes

Existing morning and afternoon peak hour volumes were collected in May 2008 at the intersections of Pinkie Road with Highway No. 1, Dewdney Avenue, 9th Avenue and Armour Road. Forecast traffic volumes were then developed by applying a linear growth rate of 1 percent per year over 25 years to derive forecast volumes for the 25-year forecast scenario. Forecast background afternoon peak hour two-way volumes on Pinkie Road are expected to range from 35 vph at Highway No. 1 to 265 vph between 13th Avenue and Dewdney Avenue.

Background forecast 25-year volumes may also include traffic volumes generated by the Southwest Sector. The following assumptions were made:

- The Southwest Sector study included an assessment of the future road network required to serve southwest Regina to two forecast population timeframes (i.e. 235,000 and 300,000)
- The model included two additional interchanges along Highway No. 1 west of Lewvan Drive – one located at Pinkie Road and a second located at Courtney Street. The traffic volumes generated by the Southwest Sector were assumed to utilize the Lewvan Drive interchange within the 235,000 population timeframe.
- Limited volumes were assigned to the Pinkie Road corridor during the 235,000 population forecast
- Volumes assigned to Pinkie Road were for the 300,000 population forecast. Volumes ranged from 100 vph at Highway No. 1 to 600 vph north of 13th Avenue

No Southwest Sector volumes are being assigned to Pinkie Road for the 25-year forecast (i.e. Phases 1 and 2), only the full growth Phase 3 scenario.

It is also anticipated that a small percentage of Highway No. 1 Bypass (i.e. Ring Road) traffic will transfer to the new west bypass to travel between Highway No. 1 and Highway No. 11. This traffic must currently use either Lewvan Drive/Pasqua Street or the Ring Road to travel between these two highways. It was assumed that 10 percent of the daily traffic volumes on those sections of the Ring Road (between Highway No. 6 south and Wascana Parkway - approximately 19,000 vpd) and on Albert Street North (north of the Ring Road - approximately 22,000 vpd) would transfer to the west bypass. This corresponds to approximately 2,000 vpd or 200 vph transferring to the west bypass.

3.6 Total Forecast Traffic Volumes

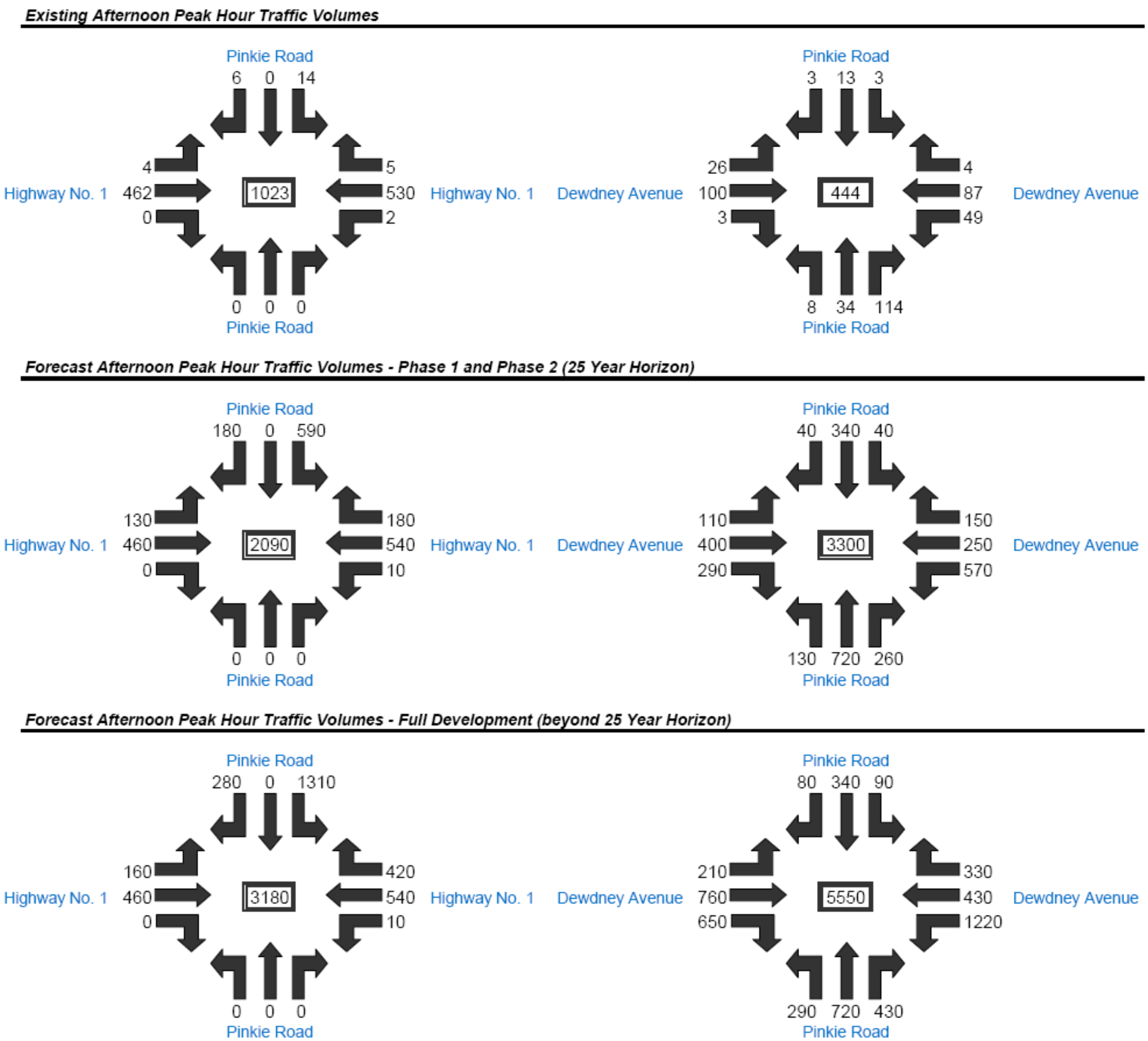
The total forecast afternoon peak hour traffic volumes for each scenario (existing, 25 year forecast with development from Phases 1 and 2, and full development beyond the 25-year horizon) and for the intersections of Highway No. 1 and Pinkie Road and Highway No. 1 and Dewdney Avenue are illustrated in Figure 1 – Existing and Forecast Traffic Volumes.

A capacity analysis was conducted on the forecast turning movement counts using Synchro and SimTraffic Version 7.0. Measures of Effectiveness (MOEs) such as Average Delay, Level of Service (LOS), Volume to Capacity Ratio (v/c) and Intersection Capacity Utilization (ICU) were summarized by intersection. Intersection LOS ranges in definition from LOS A, which provides the highest level of operational service to intersection users, to LOS F, which constitutes failure of the intersection or the turning movement being studied. LOS D is commonly considered the limit of acceptable operation. Significant delays in traffic can occur below this level.

Intersection capacity utilization and volume-to-capacity ratios were used to indicate how much reserve capacity is available or how much the intersection or specific movements are overcapacity. The intersection geometry should be designed for an ICU and v/c ratio of 0.85 or lower for each intersection/approach during peak hours. Key infrastructure impacts from the increase in traffic are:

- Highway No. 1 and Pinkie Road was analyzed as an at-grade intersection with stop control for north-south movements. The largest movement is the southbound left turn (590 vph for the 25-year forecast and 1,310 vph for the long-term +25-year forecast)
- The southbound left turn movement will operate at a LOS F (i.e. delays of over 50 seconds per vehicle) and a volume to capacity (v/c) ratio of greater than 1.0 for both the 25-year forecast and the beyond 25-year forecast. A sensitivity analysis of the southbound left turn movement indicates that even with volumes of 25-30 vph, the movement will operate at a LOS F due to the limited gaps and high eastbound and westbound volumes.
- The long delays may result in these drivers taking unacceptable risks in an attempt to enter the highway before it is safe to do so. An interchange at this location will be required prior to full development of Phases 1 and 2.

Figure 1 – Existing and Forecast Traffic Volumes



The intersection of Dewdney Avenue and Pinkie Road was analyzed as an at-grade intersection with signalization. In the long term, two northbound and southbound through lanes will be required. Single right and left turn auxiliary lanes in each direction were modelled. Key movements at this intersection include the eastbound through, northbound through and westbound left turn.

- In the long term (+25 year scenario), there could be an extremely high demand for access to Pinkie Road from the industrial developments located east of Pinkie Road. Employee traffic would utilize Pinkie Road to gain access to the southwest, south and east sectors of Regina, as opposed to using Lewvan Drive and other internal roadways. However, the amount of traffic assigned to the westbound left turn is exceptionally high (+1,200 vph), resulting in huge delays. While the demand may be high, it is anticipated that with long delays, traffic will redistribute to Courtney Street and Lewvan Drive to access the south, west and east sectors of the City of Regina. For the purposes of the analysis, a westbound left turn volume of 600 vph was used in the long-term analysis

For the 25-year forecast, there will be ample capacity at this intersection to accommodate increase in traffic, assuming auxiliary right and left turn lanes are provided for each approach, as well as dual through lanes for each approach. With this geometry, the intersection will operate at a LOS B and an ICU of 59%, with all individual movements operating at a LOS C or better

- For the beyond 25-year forecast and full development of the industrial area, there will be limited capacity at this intersection to accommodate all traffic, assuming the same geometry. The intersection is forecast to operate at a LOS E and an ICU of 93%, with several individual movements operating at a LOS F
- It is recommended that in the long term (beyond 25 years), dual northbound and westbound left turn lanes be provided. This will improve the intersection operations to a LOS C and an ICU of 77 percent. All individual movements will improve to a LOS D or better

There are other infrastructure considerations to serve the study area, including the available capacity and required cross section for Dewdney Avenue.

Service volumes for a roadway with a similar classification to the Dewdney Avenue outside of the current city limits (i.e. low access density, no parking, a 65-75 km/h speed limit, little pedestrian activity and low to medium density of roadside development) was referenced from Exhibit 10-7 from the Highway Capacity Manual (2000). Service volumes are an estimate of how many vehicles per hour a roadway can carry at a given level or service per number of lanes (per direction).

With one lane per direction, the following service volumes and corresponding levels of service (LOS) can be achieved for a 2-lane roadway:

- LOS C – 620 vph per direction per lane
- LOS D – 820 vph per direction per lane
- LOS E – 860 vph per direction per lane

The traffic volumes generated by the industrial area and assigned to Dewdney Avenue for the 25-year forecast will range from 1,600 vph east of Pinkie Road and 1,150 vph west of Pinkie Road, resulting in a LOS D and LOS B, respectively.

For the long-term (beyond 25-year) forecast, the traffic volumes will range from 3,100 vph east of Pinkie Road and 2,350 vph west of Pinkie Road, resulting in a LOS F, suggesting that a 4-lane cross section will be required in the long term.

3.7 Transportation Cost Estimates

The only road network improvement included in the costing exercise is the upgrade of Dewdney Avenue from Lewvan Drive to the IMF access road. Dewdney Avenue will be a key access to and from the City of Regina for new trips originating from the industrial area. The cost estimate assumes a strengthening of the existing 4-lane section from Lewvan Drive to Courtney Street, and the strengthening and widening to a new 4-lane cross section from Courtney Street to the IMF access road. The estimate also includes signals at the intersection of Pinkie Road and Dewdney Avenue and at the intersection of Courtney Street and Dewdney Avenue.

The planning level construction cost estimates for the recommended modifications are based on preliminary estimates of roadway materials, curb and gutter if the roadway is an urban cross section, structures, street lighting, traffic items (i.e. signals, pavement markings) and include a 25 percent allowance for engineering and allowance for construction contingency.

Table 6 – Construction Estimate identifies the estimated costs for upgrading Dewdney Avenue:

Table 6 – Construction Estimate

Description	Unit	Length	Unit Price	Total
Dewdney Avenue Upgrade				
1. Strengthen and Widen from Courtney Street to IMF Access				
• Upgrade existing Dewdney Avenue	km	4.9	$500,000	$2,450,000
• Widen to 4-lane Facility	km	4.9	$1,300,000	$6,370,000
• Intersection treatments (Channelization on Dewdney Avenue.)	Lump sum	2.0	$200,000	$400,000
2. Strengthen Existing Four Lane - Lewvan Dr. to Courtney Street				
• Upgrade existing Dewdney Avenue	km	3.1	$500,000	$1,550,000
3. Traffic Signals				
• Signalize Dewdney Avenue & Courtney Street	Lump sum	1.0	$150,000	$150,000
• Signalize Dewdney Avenue & Pinkie Road	Lump sum	1.0	$150,000	$150,000
			Subtotal	$11,070,000
			Engineering and Contingency 25%	$2,768,000
			Total	$13,838,000

It is expected that the within the first year of development (i.e. end of 2009), Dewdney Avenue will require upgrading from the IMF access road to Courtney Street. This would coincide with the construction of Pinkie Road as a 2-lane highway from Highway No. 1 to Dewdney Avenue within the next year. The construction of Dewdney Avenue to a 4-lane facility would correspond with the full development of the Loblaw's and IMF sites, as well as the 4-laning of Pinkie Road in order to provide continuity between facilities – this is anticipated within the next five years. Traffic signals at Pinkie Road and Courtney Street will be warranted as traffic demand increases.

4. Municipal Services

4.1 Storm Water Servicing

4.1.1 Existing Information

Currently, the proposed IMF and Industrial Lands development is situated on cultivated agricultural land. The general topography in the area consists of a height of land, which divides Section 21 from the southeast to the northwest. The general storm water flow patterns can be seen on Drawing 04 – Predevelopment – Storm Water Review.

The lands to the west of the divide drain to two locations. The lands in the southwest corner of the proposed annexed lands drain to an outlet directly into the Grand Coulee Drainage Ditch. The majority of the land, however, drains through a poorly defined drainage route to an outlet approximately 1.7 km north of the railway tracks. The north drainage ditch converges with the Grand Coulee Drainage Ditch in SW23-Township 17-Range 21-W2M. The flows are then routed to Cottonwood Creek, west of Grand Coulee.

The lands to the east of the drainage divide drain to Wascana Creek. The northeast portion of section 21-17-20-W2M drains via a culvert through Dewdney Avenue. into a ditch, which conveys the flow to Wascana Creek. The lands in section 28-17-20-W2M drain overland to Wascana Creek, concentrating at multiple inlet locations.

4.1.2 Topographic Information

The City of Regina currently has 0.25 metre contour information for the majority of sections 21 and 28-17-20-W2M as well as a portion of the east half of section 20. They also have contours for the lands north of Dewdney Avenue in sections 29 and 30-17-20-W2M. For the remainder of the lands topographic information was obtained from 10-metre National Topographic Survey (NTS) maps and the Government of Canada Toporama website, which generates rough 1-metre contours based on the NTS 10 metre contours.

As this study did not include fieldwork, it was agreed upon by the City and UMA that the location of culverts and drainage routes would be determined from the existing topographic information and aerial photography. As the location and size of the existing culverts was not known, it was also agreed that the existing storm water system would be modelled with culverts sized to cause minimal flow resistance at crossing locations.

4.1.3 Storm Water Modelling

The XPSWMM modelling platform was used to determine the appropriate size for the storm water channel. The model parameters used in XPSWMM were based on previous studies completed for the City of Regina and are given in Table 7 – Model Parameters.

Table 7 – Model Parameters

Component	Parameter	Value
Impervious Area Runoff	Depression Storage	6.0 mm
	Manning's "n"	0.013
Pervious Area Runoff	Depression Storage	12.0 mm
	Manning's "n"	0.25
Horton Equation	Maximum Infiltration Rate	43.2 mm/hr
	Minimum Infiltration Rate	7.6 mm/hr
	Decay Rate of Infiltration	0.00115 sec^{-1}
Percent Impervious	Residential	50%
	Industrial	80%
	Undeveloped/Agricultural	2.50%

4.1.4 Conceptual Storm Water Analysis

4.1.4.1 Storm Water Management Facilities

Based on direction from the City of Regina, the preferred method of post development storm water management for these lands is detention (dry) or retention (wet) storm water management facilities sized to detain flows greater than the predevelopment 1 in 100-year Chicago Storm event. The facilities will be allowed to drain to the existing outlet at a rate not to exceed the predevelopment 1:100 year flow.

The layout of the storm water management facilities will be based on land ownership and the facilities will be constructed and operated by individual landowners. Although the facilities will not be owned or operated by the City, i.e., regional facilities, the development will still require consideration and management of storm water flow from upstream lands. Proposed development must include a storm water management plan that accommodates the predevelopment flows of the upstream lands.

The conceptual storm water management facility layout can be seen on Drawing 05 – Post Development – Storm Water Review and is based on existing topography, landownership and conceptual infrastructure layout for the IMF and Loblaw's site.

4.1.4.2 Storm Channel

Per the City of Regina Development Standards Manual 2008 "Major storm channels shall be designed for the 1:100 year runoff flow. Side slopes shall not be steeper than 4:1. Channels shall be designed with consideration of erosion factors, gradient requirements for efficient flow, and long term maintenance aspects relating to vegetation control and periodic sediment removal and an incorporated low-flow pipe drainage system shall be considered." Based on further direction from the City of Regina, storm water channels shall have slopes no steeper than 5:1.

4.1.4.3 East Catchment – Wascana Creek

The lands to the east of the height of land in section 21-17-20-W2M currently drain north through a culvert in Dewdney Avenue and through a ditch into Wascana Creek. Providing the ditch remains post development and the storm water management facility release rate does not exceed the existing runoff rate the existing infrastructure should be adequate. If the existing culvert and ditch are inadequate to handle the predevelopment 1:100 year runoff then upgrades will be required. The capacity of the culvert crossing and ditch will need to be evaluated once a survey is complete and culvert information is gathered.

The land in section 28-17-20-W2M currently drains overland into Wascana Creek. Based on existing landownership two storm water management facilities will be required. Because of their proximity to Wascana Creek these ponds will be able to release directly into the Creek without the need of a regional ditch or storm channel. They will also be able to drain by gravity to the creek versus pumping due to the existing topography.

4.1.4.4 West Catchment – Cottonwood Creek

The lands to the west of the height of land in section 21-17-20-W2M currently drain west and exit the annexed area at two locations as shown on Drawing 05 – Post Development – Storm Water Review.

The catchment to the south contains approximately 72 hectares. Post development, the lands within this catchment will drain to the north outlet.

The catchment contributing to the north outlet contains approximately 1197 hectares and generates a predevelopment runoff rate of 5.55 m^3/s. Approximately 2.02 m^3/s of this flow is generated by lands north of Dewdney Avenue which are routed through the proposed annexed lands in the north half of section 24. The flow generated north of Dewdney will need to be accommodated post development by means of a ditch capable of conveying the 2.02 m^3/s flow. With the peak flow from the lands north of Dewdney accommodated through a north flow bypass, the available release rate for the proposed annexed lands is 3.53 m^3/s or approximately 3.36 L/s/ha (0.048 cfs/acre or 30.7 cfs/sq. mile). The release rate of 3.36 l/s/ha is reasonable for this type of prairie and takes into consideration the fact that the lands from the south catchment will be drained through the north outlet. Based on the conceptual storm water management facility layout and the release rates associated with each parcel, the storm water channel was broken into eight separate reaches and sized to accommodate the associated flows. The conceptual flows and channel properties are listed in Table 8 – Conceptual Flows and Channel Properties below. The conceptual storm water management facility and storm water channel layout are shown on Drawing 05 – Post Development – Storm Water Review.

The channel top width is from left inside shoulder to right inside shoulder and does not account for a maintenance access road, as suggested in the City of Regina Development Standards Manual.

Table 8 – Conceptual Flows and Channel Properties

Reach/Crossing	Cross Section	Flow (m^3/s)	Depth/ Diameter (m)	Width (m)	Top Width (m)	Length (m)	Gradient (m/m)
Reach 1	Trapezoidal	0.40	1.35	3	16.5	735	0.00157
Pinkie Road Culvert	One - 750 mm CSP	0.40	0.75	---	---	20	0.00157
Reach 2	Trapezoidal	0.83	1.60	3	19	740	0.00157
Proposed Highway Culvert	One - 1000 mm CSP	0.83	1	---	---	100	0.00157
Reach 3	Trapezoidal	1.05	1.8	3	21	634	0.00157
Reach 4	Trapezoidal	1.31	1.8	3	21	993	0.00157
Condie Road Culvert	One - 1200 mm CSP	1.31	1.2	---	---	20	0.00157
Reach 5	Trapezoidal	1.59	1.8	3	21	870	0.00157
Reach 6	Trapezoidal	2.26	2	3	23	1568	0.00157
IMF Access Road Culvert	Two - 1200 mm CSP	2.26	1.2	---	---	36	0.00157
Reach 7	Trapezoidal	2.69	2	3	23	664	0.00157
Reach 8	Trapezoidal	3.08	2	3	23	328	0.00153
North Flow Bypass	Trapezoidal	2.02	1.75	3	20.5	1543	0.00259

4.1.5 Storm Water Servicing Cost Estimate

Table 9 – Conceptual Storm Water Cost Estimate shows the estimated cost of the proposed storm water servicing shown in Drawing 05 – Post Development – Storm Water Review.

Table 9 – Conceptual Storm Water Cost Estimate

Reach/Crossing	Class D - Cost
Reach 1	\$130,000
Pinkie Road Culvert	\$20,000
Reach 2	\$170,000
Proposed Highway Culvert	\$70,000
Reach 3	\$450,000
Reach 4	\$570,000
Condie Road Culvert	\$20,000
Reach 5	\$240,000
Reach 6	\$520,000
IMF Access Road Culvert	\$60,000
Reach 7	\$220,000
Reach 8	\$110,000
Sub-total South Channel	**\$2,580,000**
North Flow Bypass	\$410,000
Total for N & S	**\$2,990,000**

The cost estimate for the storm channel includes an allowance for riprap erosion protection for approximately 1130 metres in reaches 3 and 4 where the 1.0 m contour lines suggest steeper slopes may be encountered.

4.1.6 Storm Water Summary

The regional storm water channel required to accommodate the predevelopment 1 in 100 year rainfall event has a conceptual design bottom width of 3.0 m in order to accommodate the equipment required to construct the channel. The channel depth ranges from 1.35 m to 2.0 m. The top width ranges from 16.5 m to 23.0 m from inside shoulder to inside shoulder. The noted widths do not include allowances for maintenance access, as suggested in the Development Standards Manual. The total Class 'D" cost estimate to construct the channel is $2,990,000. The cost estimate for the storm channel includes an allowance for riprap erosion protection for approximately 1130 metres in reaches 3 and 4 where the 1.0 m contour lines suggest steeper slopes may be encountered.

4.2 Wastewater Servicing

UMA Engineering Ltd. (UMA) was retained by the City of Regina (City) to undertake the IMF and Industrial Lands Servicing Review. The primary purpose of the wastewater servicing review was to:

- Describe the existing wastewater system in the vicinity of the proposed development and identify any major constraints
- Estimate additional flows that will be generated by the proposed development
- Estimate the expected wastewater collection system response to the additional flows under existing and future conditions
- Recommend short term and long term infrastructure improvements to allow the proposed development to proceed
- Identify the financial implications associated with providing this servicing and/or improvements to the wastewater system.

4.2.1 Existing Wastewater System and Major Capacity Constraints

Figure 2 – Regina's Critical Wastewater Infrastructure shows the existing wastewater infrastructure in the vicinity of the proposed development. As noted in the figure, the only major components that currently exist near the development are the Wastewater Treatment Plant (WWTP) and the McCarthy Booster Pumping Station (MBPS) Forcemain. No other wastewater infrastructure is available to service even a portion of the proposed site at this time.

Figure 3 – Schematic of Wastewater Treatment Plant Yard Piping shows a schematic of the WWTP site including the incoming forcemains from the east. The forcemains terminate at a valve chamber (VC) near the east limit of the facility. Wastewater from the valve chamber under normal conditions flows north towards the Primary Treatment Plant (PTP) located near the northwest corner of the site through a 1350 mm reinforced concrete conduit. A return 1350 mm reinforced concrete conduit from the PTP delivers the wastewater flows to the lagoons for further polishing before draining to the Tertiary Treatment Plant (TTP) and Ultraviolet (UV) Disinfection facilities. The treated effluent is then discharged to Wascana Creek downstream of the UV facility.

The current capacity of the WWTP is roughly 250 MLD. It is limited primarily by the capacity of the PTP facility that is in turn mostly limited by the existing capacity of the three primary effluent pumps that convey flow from the PTP to the lagoons.

The MBPS forcemain consists of two parallel 1050 mm steel conduits that flow under pressure and terminate at the valve chamber (VC) located near the west projection of 1st Avenue North and Condie Road (the east limit of Section 30 – Township 17 – Range 20) as shown in Figure 3. The forcemains are referred to as the North and South forcemains. They were installed in 1991 and 1958, respectively. The South forcemain is currently being investigated for possible replacement or refurbishment.

The existing capacity of the MBPS Forcemain is roughly 300 MLD with all pumps turned on at the station. It is limited primarily by the capacity of the existing 1350 mm conduit leading from the VC to the PTP. This conduit represents a significant hydraulic loss under the higher flow conditions experienced at the plant.

Critical systems, such as the MBPS, are typically designed with redundant pumping capacity such that the required flow rates can be provided with one of the largest pumps out of service. In the case of the MBPS, the existing firm capacity is approximately 275 MLD assuming that the forcemain discharges to one of the lagoons as opposed to the PTP and one of the large diesel driven pumps is out of service.

Figure 2 – Regina's Critical Wastewater Infrastructure

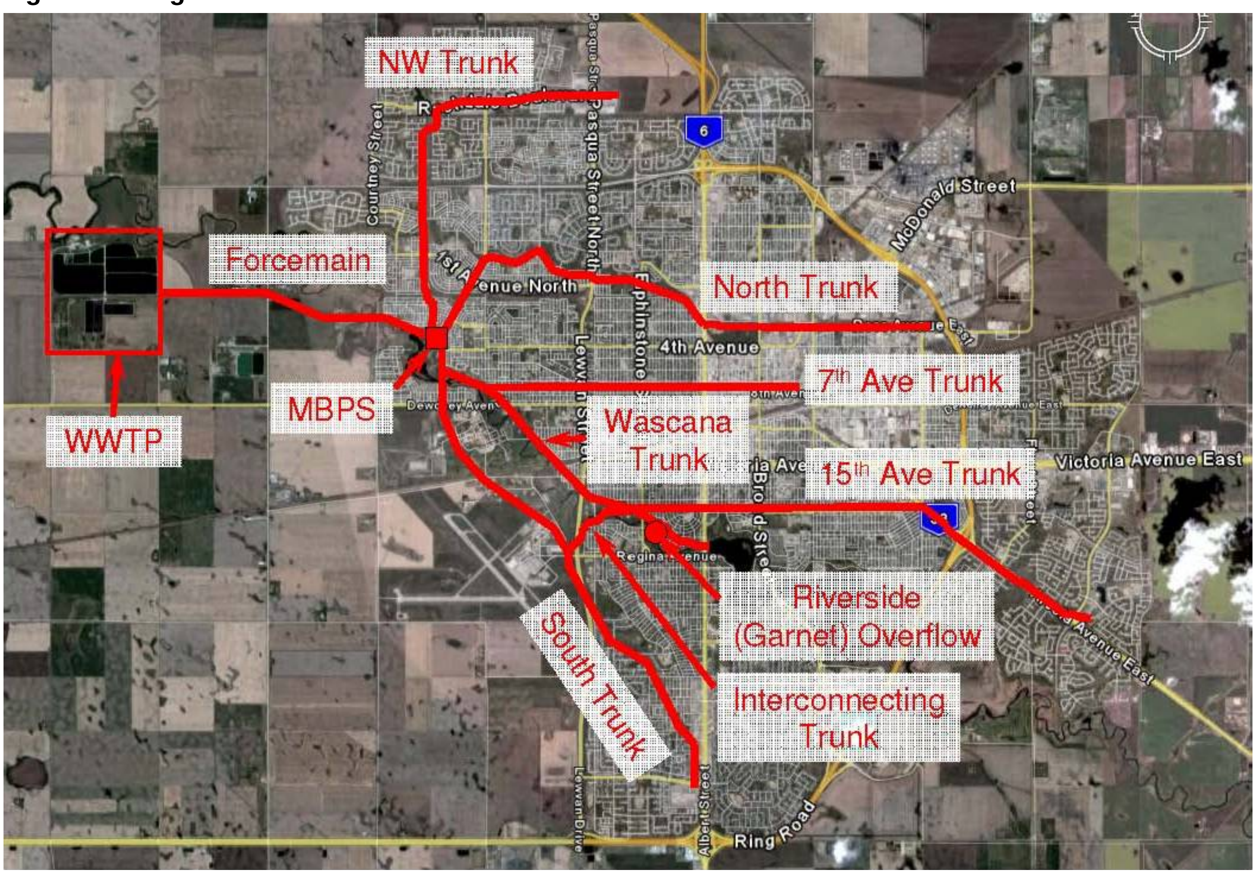
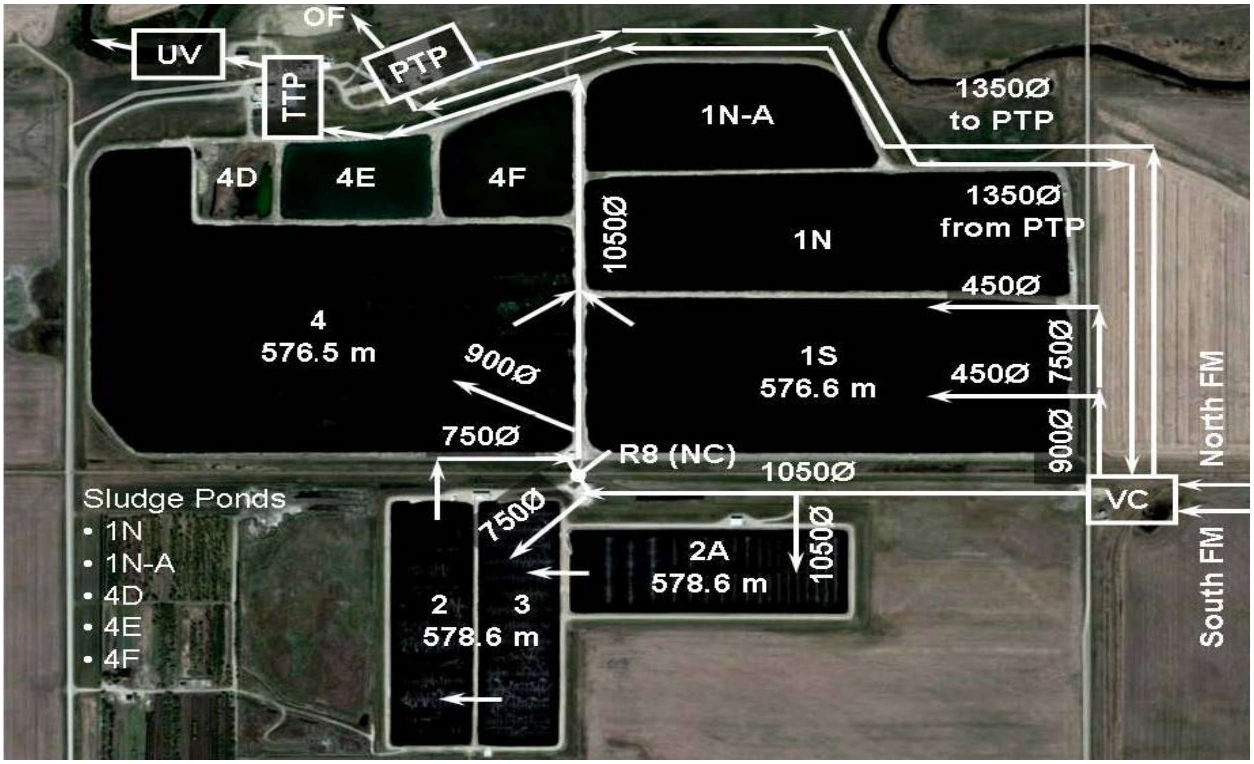


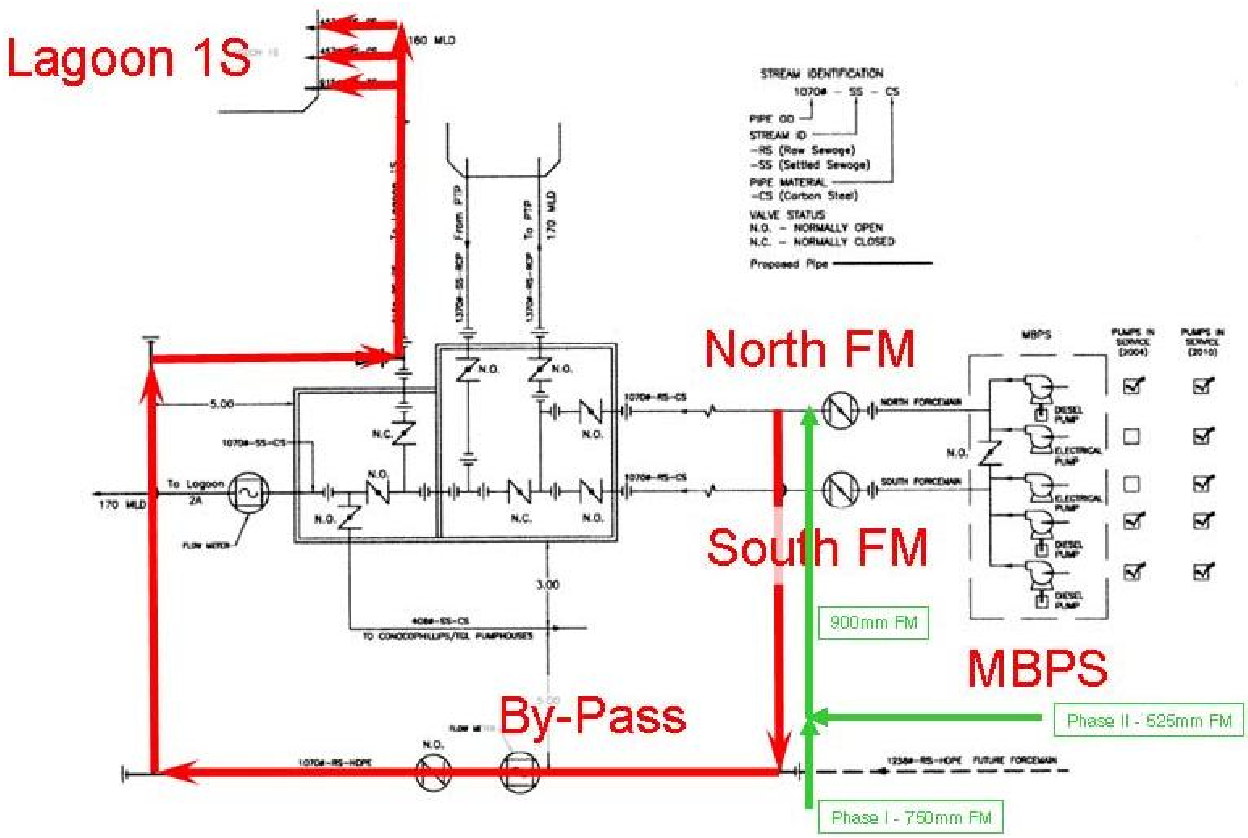
Figure 3 – Schematic of Wastewater Treatment Plant Yard Piping



Due to the lack of existing public wastewater infrastructure in the area, the servicing options for the IMF and Industrial Lands Development are generally limited to tying in to either the MBPS forcemains or to the existing valve chamber (VC) located at the southeast corner of the WWTP site.

Tying in at the VC is advantageous because the boundary conditions at this location will remain relatively constant over time, that is, the pressures will remain close to atmospheric, regardless of the incoming flows from the MBPS forcemains. Figure 4 shows a close-up schematic view of the VC location and a possible configuration of the IMF and Industrial Lands tie-ins. Tie-ins are located just outside the existing building footprint in the schematic due to the limited space available inside the existing building.

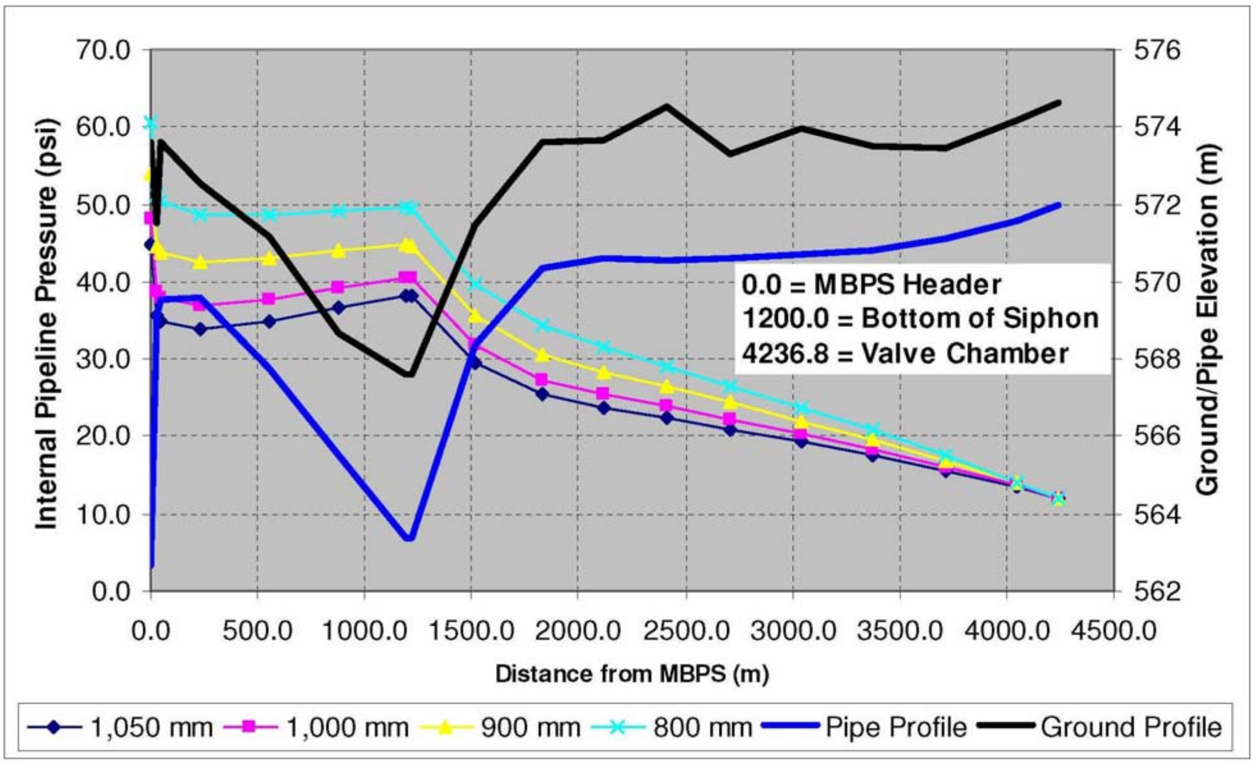
Figure 4 – Valve Chamber Location Schematic



In contrast to tying in at the VC, tying in along the forcemains will not be as advantageous because re-configuration and/or refurbishment of the forcemains or changes to the upstream loading conditions could have a significant impact on the hydraulic boundary conditions. These conditions in turn could have significant affect on the proposed infrastructure for the IMF and Industrial Lands.

Figure 5 – Internal Pressure Along South Forcemain at a Flow Rate of 300 MLD shows the pressure profile of the MBPS forcemain under various refurbishment scenarios for the South forcemain at a flow rate of 300 MLD. This flow condition uses both forcemains to convey the flows with the pressures in both being very similar (if not identical). The figure shows that the pressures at the MBPS Header (Station 0 in the figure) vary from roughly 35 psi to 60 psi depending on the internal diameter of the refurbished South forcemain. For example, a new 1050 mm forcemain will have pressure at the pump station header of 35 psi while a pipe liner with an internal diameter of 800 mm will increase the pressure at the header to 60 psi. Conversely, the pressure at the valve chamber remains at roughly 10 psi for all the selected forcemain diameters or at a hydraulic head of approximately 579 m, i.e., 10 psi = 7.03 m of water plus the pipe invert at 572 m = 579 m.

Figure 5 – Internal Pressure Along South Forcemain at a Flow Rate of 300 MLD (from McCarthy Boulevard Pumping Station 1958 Forcemain Pre-Design Report)



4.2.2 Phasing/Staging and Estimated Sewer Flows

4.2.2.1 Phasing and Staging

Section 2.0 Land Use Plan - Phasing & Staging and Drawing 03 - Phasing and Staging – Servicing Review describes in detail the proposed phasing and staging of the proposed IMF and Industrial Lands Development. MKI developed these based on discussions with various stakeholders including CP Rail and Loblaw Companies Ltd. whom together are the primary proponents of the IMF included in Phase I Stage I (see Drawing 08 – Storm Water – Wastewater - Water Phase I – Stage I). Drawing 06 –

Wastewater System – Servicing Review shows the limits and extents of the proposed wastewater servicing for the entire development.

Phase I includes the area west of Pinkie Road including the proposed IMF. Phase I has been broken up into three stages with the first stage, i.e., Stage I, expected to develop within the next 1 to 3 years. Stage II is expected to develop roughly between 3 to 25 years from now while Stage III is expected to develop beyond the 25 year planning horizon. Similar to Stage III, Phase II is expected to develop beyond the 25 year planning horizon.

The Long Term Residential Growth Study[2] suggests that the City of Regina will have a population of roughly 235,000 within the next 25 to 40 years. This growth scenario is roughly equivalent to the timeframe estimated for the full development of the IMF and Industrial Lands Development.

4.2.2.2 Estimated Wastewater Sewer Flows

Initial wastewater sewer flows for the proposed development were developed based on the City of Regina's Development Standards Manual[3], which uses the Harmon Peaking Factor (HPF) method to estimate static peak flows for the proposed system.

The components of wastewater sewer flow are commonly broken down into:

- Base System Flow (BSF),
- Groundwater induced infiltration (GWI), and
- Stormwater induced inflow (SWI).

Base System Flow (BSF)

Base system flows (BSF) according to the standards are to be estimated on a per capita basis for equivalent residential development using an average flow demand of 225 L/capita/day and future zoning floor space ratios (F.S.R's). The Development Standards suggest using F.S.R.'s of one person per 55 square meter of floor space in detached dwelling residential areas, one person per 35 square meter of floor space in apartments and one person per 23 square meter in commercial buildings.

Future zoning F.S.R. areas were not available for the IMF and Industrial Lands Development therefore a different means of estimating the wastewater flows was required. This estimate was accomplished by using information available in Drawing 02 – Land Uses – Servicing Review which is summarized in Table 1 – Land Use Schedule and Table 2 – Land Use Design Population Densities. Drawing 02 was developed based on information provided by MKI. Table 2 was developed from Table 4.3.5 which is included in Section 10 of the City of Regina Development Standards Manual. Table 4.3.5 is a list of suggested design population densities versus proposed land use zoning and was provided in the Standards to estimate water demands which is a similar process for developing wastewater base flows. By combining the information provided in Table 1 and Table 2 we were able to develop equivalent residential populations for the various phases and stages noted in Drawing 03.

[2] *City of Regina. (2004). "Long Term Residential Growth Study".*
[3] *City of Regina, "Development Standards Manual, Section 8.0 Domestic Sewer Design Standards"*

These base flows are peaked using the Harmon's Peaking Factor (HPF) as noted below.

$$HPF = 1 + \frac{14}{4 + \sqrt{P}}$$

Where: HPF = Harmon's Peaking Factor (dimensionless)
P = Population (in 000's)

The areas that were identified as Dry Industrial in Table 1 and Drawing 02 – Land Uses – Servicing Review assumed an equivalent residential population density of 25 people/ha. Mixed industrial assumed 35 people/ha.

Table 10 – Summary of Base System Flows shows a summary of the base flows estimated for the contributing areas that are part of the various phases and stages depicted in Drawing 06 – Wastewater System – Servicing Review. We can see that the peaked base flows for the complete development, i.e., Phase I and Phase II, add up to approximately 24 MLD (14.13 + 9.72 = 23.85 MLD)

Table 10 – Summary of Base System Flows

Stage	Approx. Contributing Area (Ha)	Equivalent Population	BSF (MLD)	Harmon Peaking Factor	Harmon BSF Peak Flow (MLD)
Phase I Stage I	183	4,569	1.03	3.28	3.37
Phase I Stage I & II	666	17,907	4.03	2.70	10.88
Phase I Stage I, II & III	930	24,490	5.51	2.56	14.13
Phase II	447	15,649	3.52	2.76	9.72

Groundwater Induced Infiltration (GWI) and Stormwater Infiltration (SWI)

The GWI and SWI components of wastewater flow are added to the peaked BSF on an area basis using a value of 31,100 L/ha/day. This value is used for developments where weeping tiles are discharged directly to the wastewater sewer which is the current minimum design standard in the City of Regina. An alternative GWI and SWI flow rate parameter is 21,000 L/ha/day, which is reserved for developments that have sump pumps discharging the weeping tile flows to ground surface. The practice of surface discharge of weeping tile flows is a common practice in other jurisdictions and is done primarily to reduce SWI induced flows into the wastewater system.

Process Flows

In addition to the GWI and SWI flows, the engineering consultant for the proposed Loblaws development located in the southwest quadrant of Section 19, also provided a process flow estimate of 8 L/s. Since the currently proposed facility will only take up roughly half of the quarter section parcel, this value was doubled up to 16 L/s for the entire quarter section. A similar process flow of 16 L/s was assumed for the southeast quarter of Section 24. The combined flow of 32 L/s is equivalent to 2.8 MLD as noted Stage II and Stage III in Table 11 – Summary of Total System Flows under the Process Flow column for the Phase I area.

Total System Flows

Table 11 – Summary of Total System Flows provides a summary of the estimated total system flows for the various phases and stages using the flow parameters described above. The total estimated peak wastewater flow for Phase I Stage I is 10.5 MLD, and for Phase I Stage III the flow is 45.8 MLD. For Phase II the total estimated flow is 23.6 MLD. The total flow from both Phase I and Phase II is 69.4 MLD, which represents roughly 28% of the current WWTP capacity of 250 MLD. The extent too which current WWTP capacity is utilized and under what conditions the system is over utilized will be discussed in more detail in the following section.

Table 11 – Summary of Total System Flows

Stage	Approx. Contributing Area (Ha)	Harmon BSF Peak Flow (MLD)	Process Flow (MLD)	SWI + GWI (MLD)	SWI + GWI + Process (MLD)	Total Peak System Flow (MLD)
Phase I Stage I	183	3.37	1.4	5.7	7.1	10.5
Phase I Stage I & II	666	10.88	2.8	20.7	23.5	34.4
Phase I Stage I, II, & III	930	14.13	2.8	28.9	31.7	45.8
Phase II	447	9.72	0.0	13.9	13.9	23.6

The flows summarized in Table 11 – Summary of Total System Flows were used to size the internal wastewater components depicted on Drawing 06 – Wastewater System – Servicing Review for the IMF and Industrial Lands Development. Detailed wastewater design spreadsheets summarizing the wastewater sewer analysis for the separate phases and stages are included in Appendix B.

4.2.3 Expected System Response to IMF and Industrial Lands

Based on the flow estimates noted above, the proposed IMF and Industrial Lands Development will have a significant impact on large portions of the existing wastewater sewer infrastructure, particularly following the first stage of development. The largest impact will likely be felt at the WWTP, which has an existing capacity of between 250 MLD to 300 MLD. As indicated in the previous section, the peak static flows are estimated to be as high as 69.4 MLD for the entire development or 28% of the existing WWTP capacity. According to the recently completed McCarthy Booster Pumping Station (MBPS) 1958 Forcemain Pre-design Report[4], the WWTP already receives flows in the range of 250 to 300 MLD under extreme rainfall conditions. For example, the MBPS forcemain receives roughly 250 MLD under a 10-year rainfall event and 287 MLD under a 25-year rainfall event under existing loading conditions.

The other major impact will likely occur within the interceptor and secondary sewer system located in the southwest quadrant of Regina. This area has been identified in previous studies as an area with limited

[4] *City of Regina. (2008). "McCarthy Booster Pumping Station 1958 Forcemain Pre-Design Report".*

capacity that is susceptible to basement flooding during extreme rainfall events. The system includes the following interceptor trunk and secondary sewers:

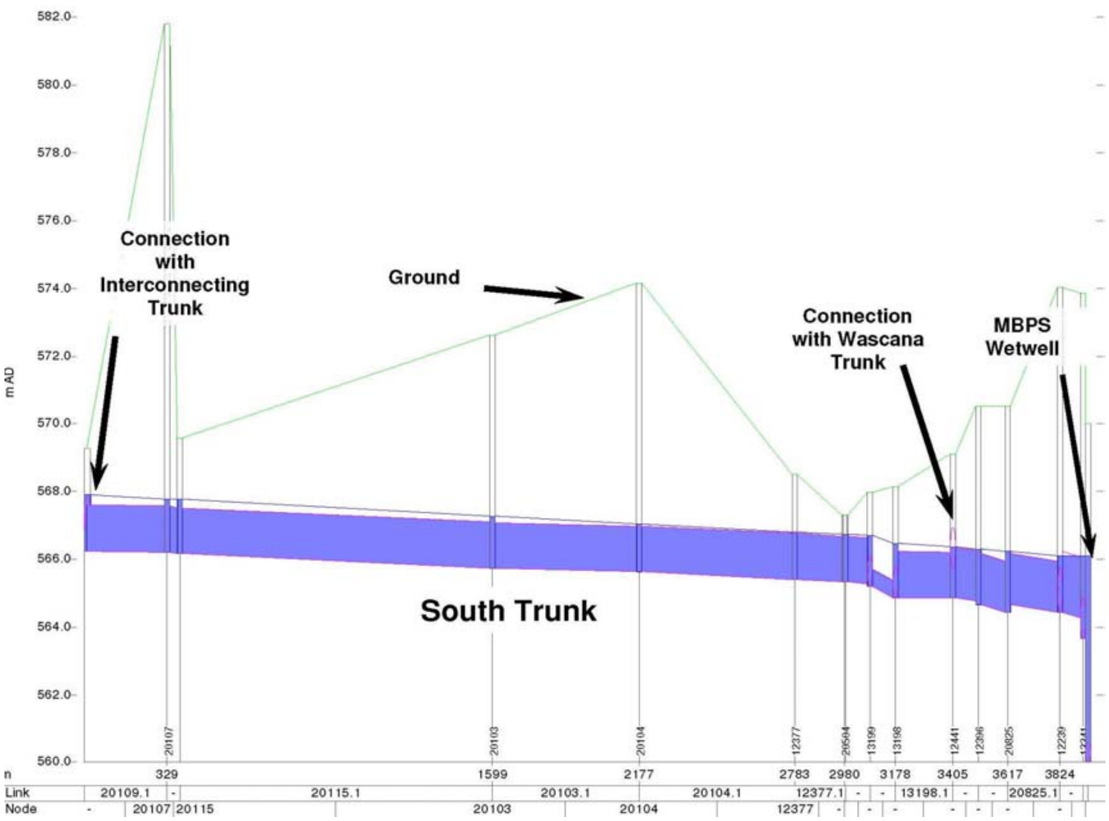
- South trunk,
- Wascana trunk, and
- South/Wascana Interconnecting trunk

Under extreme flow conditions, the South, the Wascana, and the Interconnecting trunk sewers function as a network, largely due to an undersized siphon situated near the north limit of the South trunk. When excessive flow levels are experienced in the South Trunk, the interceptor network transfers large quantities of flow to the Wascana Trunk via the South/Wascana Interconnecting Trunk. Under these extreme flow conditions, the flow levels and transfers of flow can cause basement flooding in the Southwest Sector and wastewater overflows at the Riverside (Garnet) Overflow. See Figure 2 – Regina's Critical Wastewater Infrastructure for a bird's eye view of the wastewater interceptor components located in the southwest quadrant.

4.2.3.1 Existing Baseline Conditions in SW Sector Interceptor System

Figure 6 – Hydraulic Profile of South Trunk for Existing System Under Existing Level of Development In Response to a 10-Year Event shows the hydraulic grade line profile of the South Trunk under a 10-year design storm event. As noted in the figure the existing system surcharges to roughly 0.3m above the top of the pipe at the upstream end of the interceptor system which is located near the Interconnecting trunk.

Figure 6 – Hydraulic Profile of South Trunk for Existing System Under Existing Level of Development In Response to a 10-Year Event



In response to the 25-year event the existing system generates an overflow at Garnet Street and results in a significant level of surcharging in the South Trunk (1.3 m) as shown in Figure 7 – Hydraulic Profile of South Trunk for Existing System Under Existing Level of Development In Response to a 25-Year Event. The profile suggests that the HGL daylights at Wascana Creek along McCarthy Boulevard, however, the HGL actually indicates hat the pipe has pressurized above the surrounding creek ground level at this location.

Figure 7 – Hydraulic Profile of South Trunk for Existing System Under Existing Level of Development In Response to a 25-Year Event

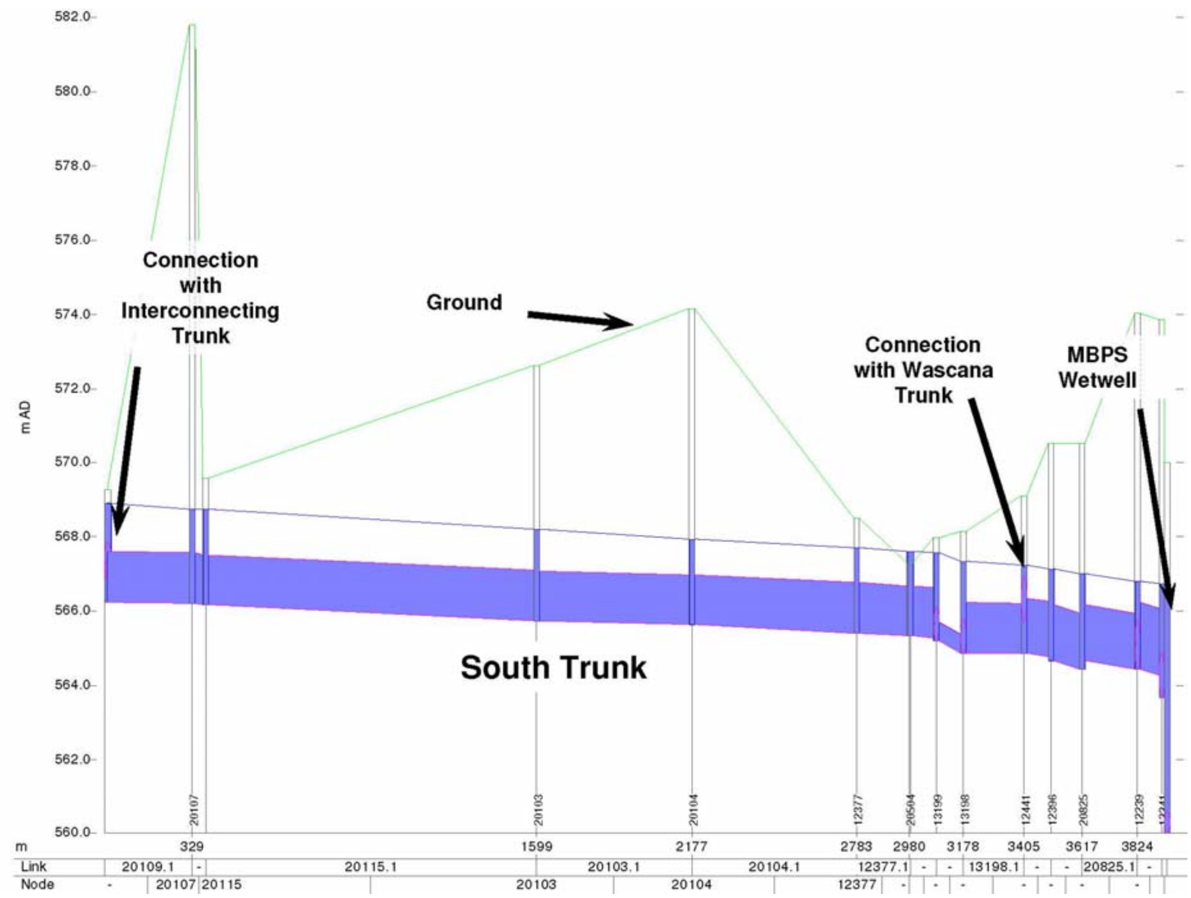


Figure 8 – Hydraulic Profile of Wascana Trunk for Existing System Under Existing Level of Development In Response to a 10-Year Event shows the hydraulic grade line profile of the Wascana Trunk under a 10-year design storm event. As can be noted in the figure there is only a small section of surcharging near the MBPS Wet Well.

In response to the 25-year event, the Wascana Trunk is surcharged throughout its length as shown on Figure 9 – Hydraulic Profile of Wascana Trunk for Existing System Under Existing Level of Development In Response to a 25-Year Event. The Garnet overflow is shown as a small circle near the left hand side of the figure. The hydraulic grade line flattens out at the top end due to the effects of the overflow which is set at an elevation of 569.3 m.

Figure 8 – Hydraulic Profile of Wascana Trunk for Existing System Under Existing Level of Development In Response to a 10-Year Event

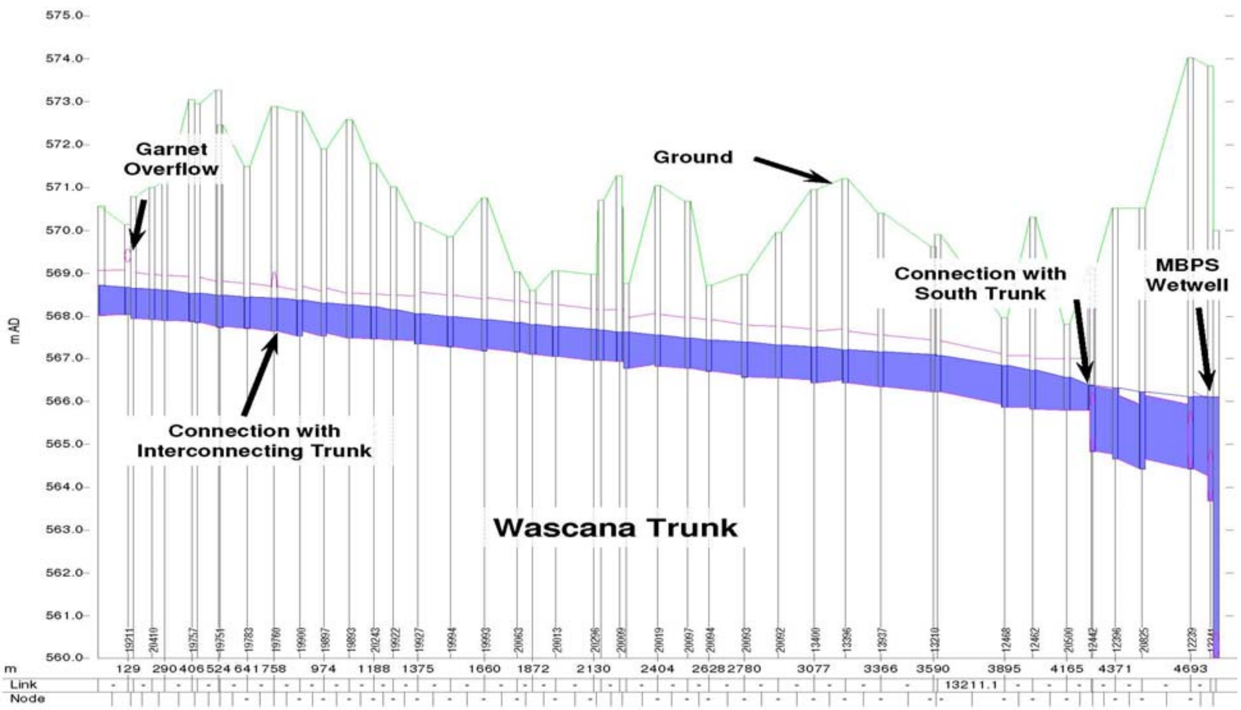
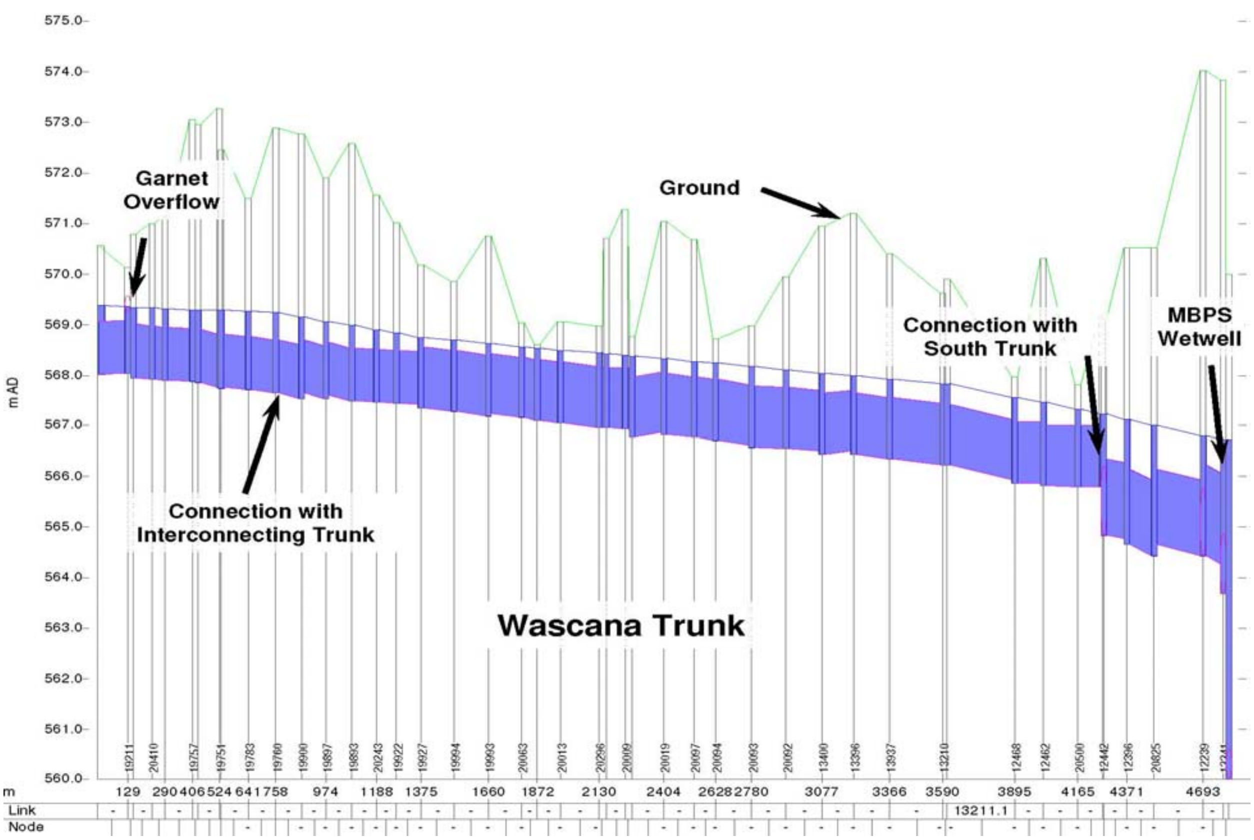


Figure 9 – Hydraulic Profile of Wascana Trunk for Existing System Under Existing Level of Development In Response to a 25-Year Event



4.2.3.2 Diurnal Flow Estimates

To truly analyze the impact of the IMF and Industrial Lands Development on the existing regional systems, it was necessary to develop flows based on a diurnal flow pattern for input into the regional models versus the static flows developed in the previous section. It is important to understand the difference between static and dynamic analytical methods as well as where and when each method of analysis should be employed. Pipe storage and routing effects are critical considerations for a rational assessment of major infrastructure components and existing system capacity, while the design of internal subdivision pipe components such as those developed for the IMF is intended to be more conservative and is reasonably accomplished by conventional static analysis.

In the City of Regina, Wastewater Collection System Study[5] each component of wastewater flow was determined independently through flow monitoring. Base flow values were derived diurnally for each land use type and are documented with diurnal peaking factors. GWI and SWI components were derived independently as well with the SWI component being broken down into storm frequency associated with each level of observed SWI (e.g. 5-year, 10-year, and 25-year levels of SWI).

Aside from the ability to use the flow output in a dynamic routing model, the diurnal peaking factors derived in the Wastewater Collection System Study recognize an important facet of wastewater hydraulic characteristics in that different land use types typically exhibit markedly different diurnal peaking patterns. For example, commercial and industrial wastewater flows rarely experience peak flows at the same time in the day as residential flows. The offset peaks are clearly evident in previous flow monitoring carried out in the City of Regina and have a pronounced impact on total peak flow projections in any scenario that has a significant component of commercial and/or industrial development.

Figure 10 – Industrial and Residential Diurnal Curves shows the weighted peak factors associated with Industrial and Residential land use in the City of Regina. From the curves, we can see that residential land use peaks early in the day (8:00 a.m.) and then tapers off quite sharply during the late morning and early afternoon periods before picking up again in the evening. Industrial land use on the other hand peaks in the early afternoon and stays relatively high until the late afternoon after which it drops off significantly through the evening and night-time periods.

The offset diurnal peaks between land use types have a significant impact on total flow predictions, particularly when utilizing hydrograph method routing analysis techniques. The diurnal patterns are based on real monitoring data from various local land use types and are valid observations for typical residential, commercial or industrial wastewater usage patterns.

In order to determine the expected system response of the existing and future wastewater systems, diurnal flows were developed for the various phases and stages for the IMF and Industrial Lands using the Industrial Diurnal Curve shown in Figure 10. The SWI and GWI flows were estimated using the 31,100 L/Ha/day parameter.

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[5] Cochrane Engineering Ltd., "City of Regina, Study of Wastewater Collection System", January 2004

Figure 10 – Industrial and Residential Diurnal Curves

4.2.4 InfoWorks Modeling

The IMF and Industrial Lands Servicing Review was conducted using the InfoWorks CS modeling platform developed by Wallingford Software. The InfoWorks platform was chosen primarily because of its superior modeling capabilities. It is able to model complex flow conditions such as surcharged pipe flow, reverse flow and storage routing and includes several convenient tools for input of various wastewater loading conditions such as the diurnal flow patterns and stormwater or groundwater induced flows described in the previous section.

InfoWorks can also be readily expanded into a citywide wastewater sewer collection system model incorporating most (if not all) of the collection system elements that currently reside in the system. Following is a brief description of the InfoWorks model that was used to analyze the expected system response of the proposed IMF and Industrial Lands Development on the existing and future wastewater collection system.

4.2.4.1 Origin and Description of Collection System Model

The existing wastewater collection system was initially analysed in 2004 by Cochrane Engineering Ltd. and Kerr Wood Leidal (KWL) Associates Ltd. with the results documented in the “City of Regina – Study of Wastewater Collection System, January 2004” report. KWL and Cochrane Engineering used the HYDRA model to analyze the City’s entire collection system.

As mentioned previously each component of wastewater flow was determined independently through flow monitoring. Base flow values were derived diurnally for each land use type. GWI and SWI components were derived independently as well with the SWI component being broken down into storm frequency, e.g. 5-year, 10-year, and 25-year levels of SWI.

Subsequent studies investigating servicing requirements for development in Southwest[6] and Northwest[7] Sectors of the City used components of the original HYDRA model. These components were first converted or imported into other modeling platforms in order to allow analysis under more complex flow conditions.

4.2.4.2 Southwest Sector Wastewater Servicing Analysis

The Southwest Sector Wastewater Servicing Analysis was performed using the EPA SWMM modeling platform. The underground conveyance system was analyzed using the Extran Module of EPA SWMM. The Extran module solves the complete St. Venant equations and is therefore capable of analyzing complex flow conditions such as storage routing, reverse flows and backwater effects.

Analysis of the wastewater system indicated that the existing trunks in the SW Sector area had the capacity to convey SWI from a 5-year storm in addition to BSF and GWI. However, in response to SWI from 10-year and 25-year storms the South, Wascana, and Interconnecting Primary Trunks and the Pasqua Secondary Trunk surcharged such that widespread basement flooding in the SW Sector was likely, particularly under the 25-year event (See Figure 6 to Figure 9).

Four alternatives, shown in Figure 11 – Southwest Sector Wastewater Servicing Options, were considered to accommodate development in the SW Sector as follows:

- Construct a new relief trunk sewer from the South and Pasqua trunks to the WW with terminal pumping at the WW.
- Connect to the existing system at select locations and utilize in-line storage to limit discharge during periods when downstream levels are greater than the 5-year SWI level.
- Provide a new gravity wastewater sewer from the new development to the WW and provide terminal pumping at the WW.
- Develop a pumping station on-site from the new development to the WW with staged pumping and forcemain capabilities corresponding to development progress.

Development in the SW Sector was phased into three stages as shown in Figure 11 and summarized in Table 12 – Phasing of Development in the SW Sector. Completion of SW Sector stages 1 and 2 corresponds to the 235,000 level of development as described in the Long Term Residential Growth Study. Stages 1 and 2 would accommodate approximately 40% of the growth required to reach the 235,000-population target. Completion of all three SW Sector stages would correspond to the 300,000 level of development and would accommodate approximately 35% of the growth required to reach that population target.

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[6] *UMA Engineering Ltd. (2006). "City of Regina – Southwest Sector Wastewater Servicing Analysis – Final Report."*
[7] *UMA Engineering Ltd. (2008). "City of Regina – Northwest Sector Servicing Study – Draft Report"*

Base system flows were estimated on a per capita basis for residential development at 225 L/capita/day and future zoning floor space ratios (F.S.R.'s) for commercial and industrial development of one person per 23 m^2. The approximate zoning floor space areas for both commercial and industrial development were provided by a previous planning study for the area and were therefore available for use in the SW Sector base system flow estimates.

Diurnal flow patterns were applied to the base system flows using the information provided from the HYDRA model. The SWI and GWI components of wastewater flow were added to the peaked BSF on an area basis of 21,000 L/Ha/day. This static value corresponded to development where weeping tiles discharge to the ground surface and are therefore not connected to the wastewater system. This value also corresponds to what is reported to be a 25-year level of service for combined infiltration (GWI) and inflow (SWI).

Table 12 – Phasing of Development in the SW Sector

Stage	Area (ha)	Equivalent Population	SWI + GWI (MLD)	BSF (MLD)	SWI + GWI + BSF (MLD)	Peak Flow (MLD)
1	170	7,195	3.6	1.6	5.2	7.3
2	240	10,757	2.4	5.0	7.4	11.2
3	520	20,504	10.9	4.6	15.5	22.9
Total	**930**	**38,456**	**16.9**	**11.2**	**28.1**	**41.4**

The servicing options for the SW Sector were evaluated and ranked based on their external servicing costs as shown in Table 13 – External Servicing Cost Estimates for the SW Sector and for their ability to be developed in a staged manner. In-line storage (Option 2) using oversized trunk sewers was the least expensive alternative and was the most amenable to a phased approach. The second least expensive alternative, forcemain by-pass with internal pumping (Option 4), did not lend itself to staged development of the SW Sector. Options 1 and 3, which involved construction of long lengths of large diameter gravity sewers and terminal pumping at the WW, were the most expensive.

The 2004 Study of the Wastewater Collection System initially proposed Option 1, as a means to provide relief to the existing SW quadrant interceptor system. However, the connection point of the relief element to the existing system at a location immediately south of Dewdney Avenue proposed in the 2004 study was found to be ineffective in the SW Servicing Study. The SW Sector Servicing Study found that the relief had to be extended approximately 3 km upstream to where the South and Interconnecting Trunks meet. In addition, the original cost estimate for this relief sewer from the 2004 study of $17,500,000 was significantly less than the updated cost estimate of $55,600,000 from the 2006 SW Servicing Study.

Figure 11 – Southwest Sector Wastewater Servicing Options

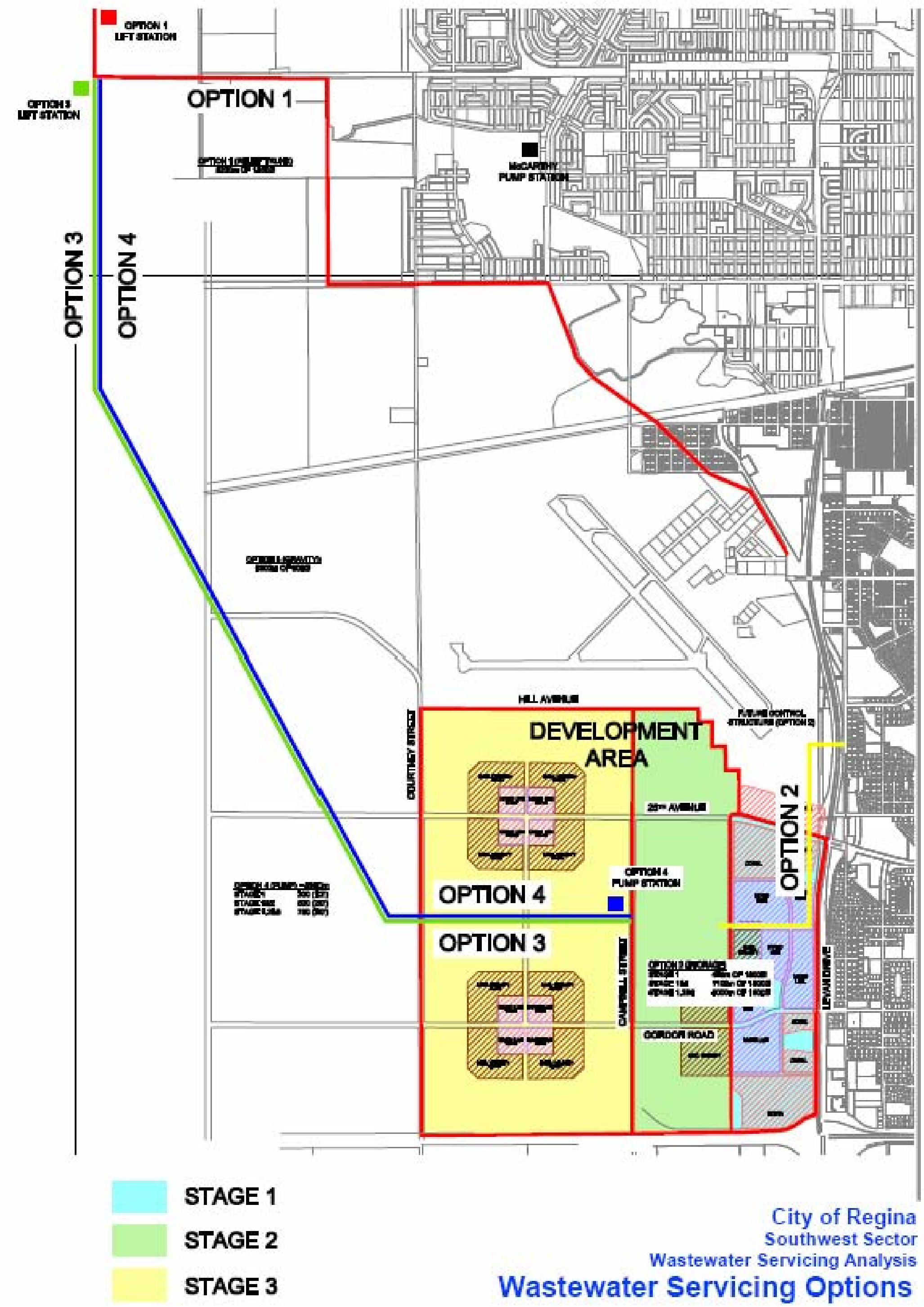


Table 13 – External Servicing Cost Estimates for the SW Sector

Option		Construction ($M)	Engineering ($M)	Contingency ($M)	Total ($M)
1		$41.2	$4.1	$10.3	$55.6
2	Stage 1	$1.8	$0.2	$0.4	$2.4
	Stage 1 & 2	$4.3	$0.4	$1.1	$5.8
	Stage 1, 2 & 3	$9.8	$1.0	$2.4	$13.2
3		$39.6	$4.0	$9.9	$56.5
4		$12.7	$1.3	$3.2	$21.2

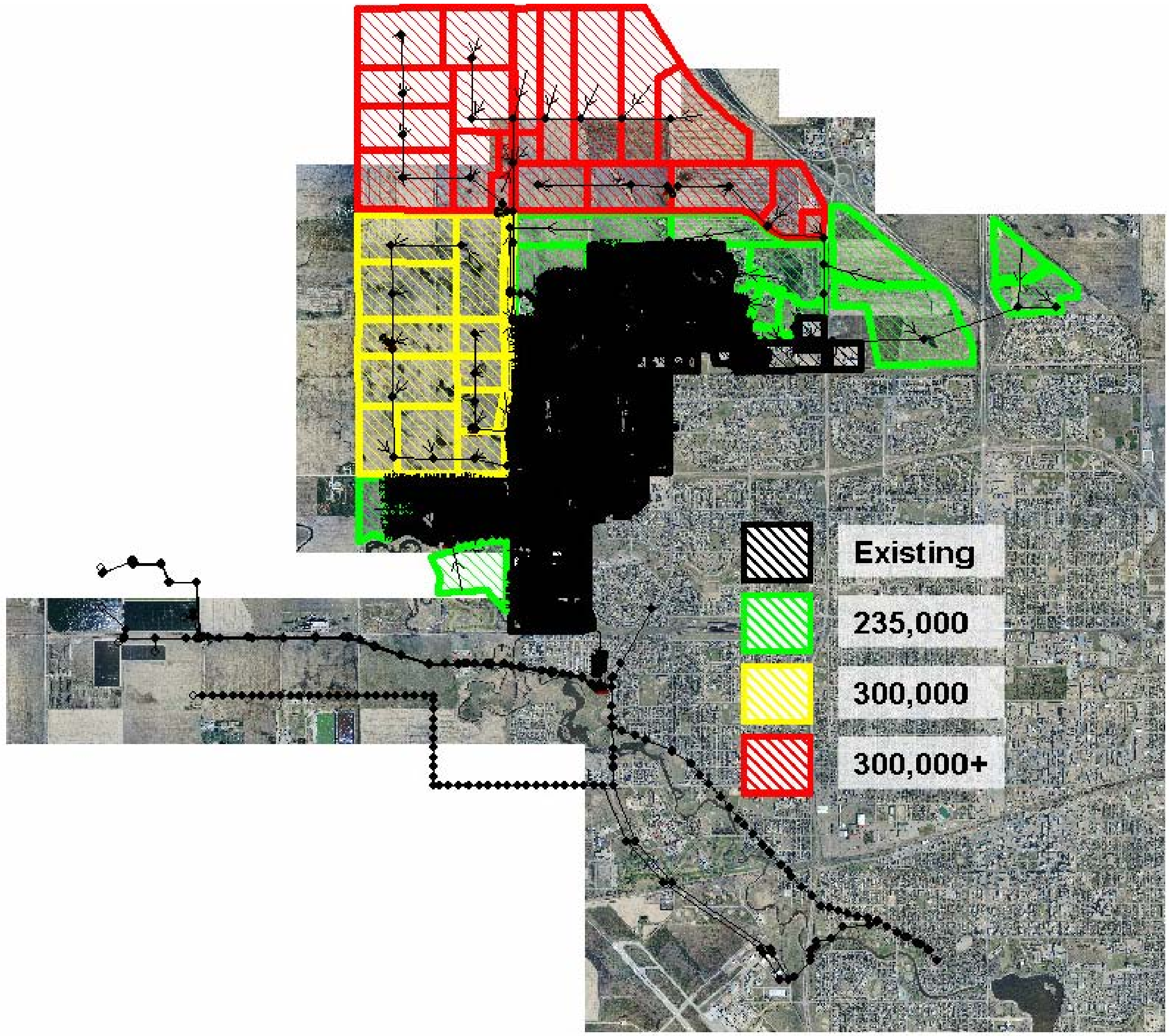
4.2.4.3 Northwest Sector Wastewater Servicing Analysis

Similar to the SW Servicing Study, the Northwest Sector Wastewater Servicing Analysis was performed using software capable of modeling complex flow conditions. In this case, however, the modeling software was the InfoWork CS modeling platform. A skeletal model of the trunk sewers leading to the MBPS was created and combined with a detailed NW Sector model. The NW sector model itself was derived from the original HYDRA model of the City's wastewater sewer collection system.

Inflows from the northeast portion of the City were generated in the HYDRA model and then added to the InfoWorks model in the form of an inflow hydrograph. In a similar fashion, inflows from the SW and SE Sectors of the City were obtained from the EPA SWMM model developed for the SW Sector Servicing Study.

Analysis of the existing NW Sector system indicated that there was excess capacity in the upper reaches of the NW trunk in response to a 25-year design rainfall event. The lower portions of the NW trunk were surcharged under a 25-year event primarily because of backwater from the MBPS. In comparison to the SW Sector where significant deficiencies in the trunk system and basement flooding concerns were identified, deficiencies in the NW Sector were fairly localized with surcharging mostly occurring within lateral sewers during large wet weather events.

Figure 12 – Northwest Sector Wastewater Servicing Options



Development in the NW Sector was phased into three stages as shown in Figure 12 – Northwest Sector Wastewater Servicing Options and summarized in Table 14 – Phasing of Development in the NW Sector below. Completion of NW Sector Stage 1 corresponds to the 235,000 level of development as described in the Long Term Residential Growth Study. It would accommodate approximately 40% of the growth required to reach that population target. Completion of NW Sector Stage 2 corresponds to the 300,000 level of development and would accommodate approximately 35% of the growth required to reach that population target. Stage 3 corresponds to development beyond the population target of 300,000.

Similar to the SW Servicing Study, base system flows were estimated on a per capita basis for residential development at 225 L capita/day. Population equivalents were estimated for the proposed industrial areas using the population densities available in the Development Standards. Diurnal patterns were applied to the base system flows. In accordance with the City of Regina's Development Standards Manual the SWI and GWI components of wastewater flow were added as static flows to the peaked BSF on an area basis of 31,100 L/ha/day. This static value corresponded to development where weeping tiles discharge directly to the wastewater system, i.e., they do not discharge to ground surface using a sump pump system as assumed in the SW Servicing Study.

Table 14 – Phasing of Development in the NW Sector

Stage	Area (Ha)	Equivalent Population	SWI + GWI (MLD)	BSF (MLD)	SWI + GWI + BSF (MLD)	Peak Flow (MLD)
1	396	16,877	12.3	3.8	16.1	19.8
2	427	18,792	13.2	4.2	17.5	21.3
3	815	35,876	25.3	8.1	33.4	40.7
Total	**1,638**	**71,545**	**50.8**	**16.1**	**67.0**	**81.8**

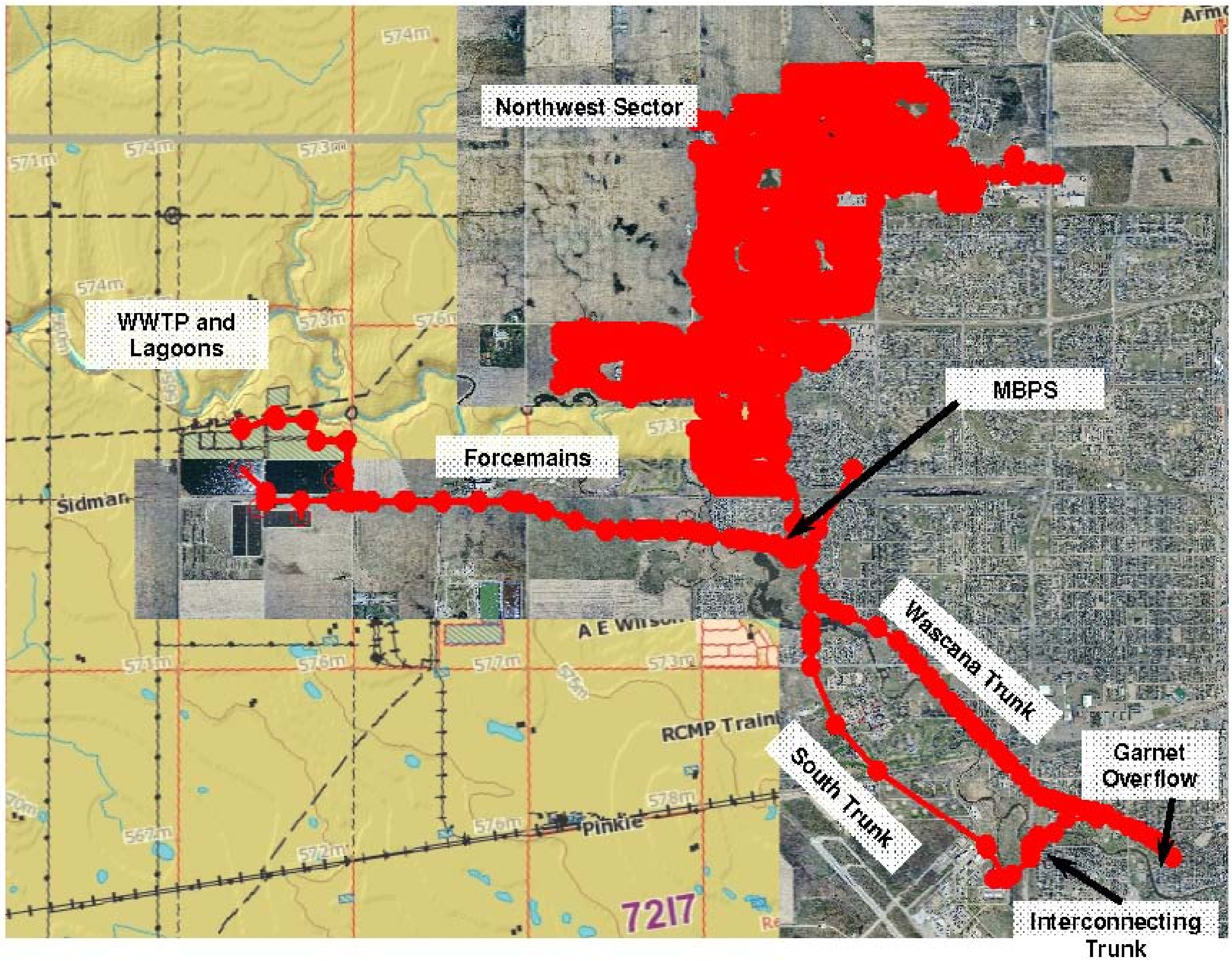
According to the Study's findings, the Stage 1 (235,000 target-population) level of development in the NW Sector can be accommodated by existing infrastructure without any adverse impact on existing service levels. This conclusion is in contrast to the SW Servicing Study that indicated some level of relief will be required to accommodate development up to the 235,000 target-population level. The section dealing with the expected future system response to the IMF and Industrial Lands Development will be further discussed in Section 4.2.5.1 below.

4.2.5 Conceptual Wastewater Analysis

The skeletal InfoWorks model used in the McCarthy Booster Pumping Station (MBPS) 1958 Forcemain Pre-design Report shown in Figure 13 – Skeletal Model of the City of Regina's Sanitary Collection System was used to analyze the expected system response with the IMF and Industrial Lands Development added to the system. The collection system model included a portion of the skeletal model used for the SW Servicing Study analysis and the detailed InfoWorks model used for the NW sector servicing analysis. It also included the components of the MBPS forcemain described in the MBPS 1958 Forcemain Pre-Design Report up to the WW.

In the SW Servicing Study, the skeletal model included the Wascana trunk up to the Garnet Street (a.k.a. Riverside) overflow, the South trunk up to and including the South/Wascana Interconnecting trunk and the Interconnecting trunk. The overflow level at the Garnet overflow was set to 569.3 m. The pumped by-pass was also configured at the MBPS which was triggered when levels exceeded 568.0 m. The forcemain was configured to discharge to the PTP via the single 1,350 mm (54") diameter feed from the valve chamber to the PTP for all flow rates.

Figure 13 – Skeletal Model of the City of Regina's Sanitary Collection System



4.2.5.1 Expected System Response With Phase I Stage I of IMF Added

Figure 14 – Skeletal Model with IMF Phase I Stage I Added shows a plan view of the skeletal model that was developed to analyze the impacts that result from the first stage of development of the IMF. The figure shows the locations of the IMF and the proposed Loblaws development relative to the existing wastewater infrastructure in the vicinity. It also shows the approximate location of the proposed Phase I Pump Station (PS) and forcemain that will be required to service the Stage I development.

See Drawing 06 – Wastewater System – Servicing Review for a schematic of the proposed wastewater conduit sizes assumed for the full development of the IMF and Industrial Lands and Drawing 08 – Storm Water – Wastewater – Water Phase I – Stage I for a close up view of the proposed municipal servicing scheme for Phase I Stage I.

Figure 14 – Skeletal Model with IMF Phase I Stage I Added

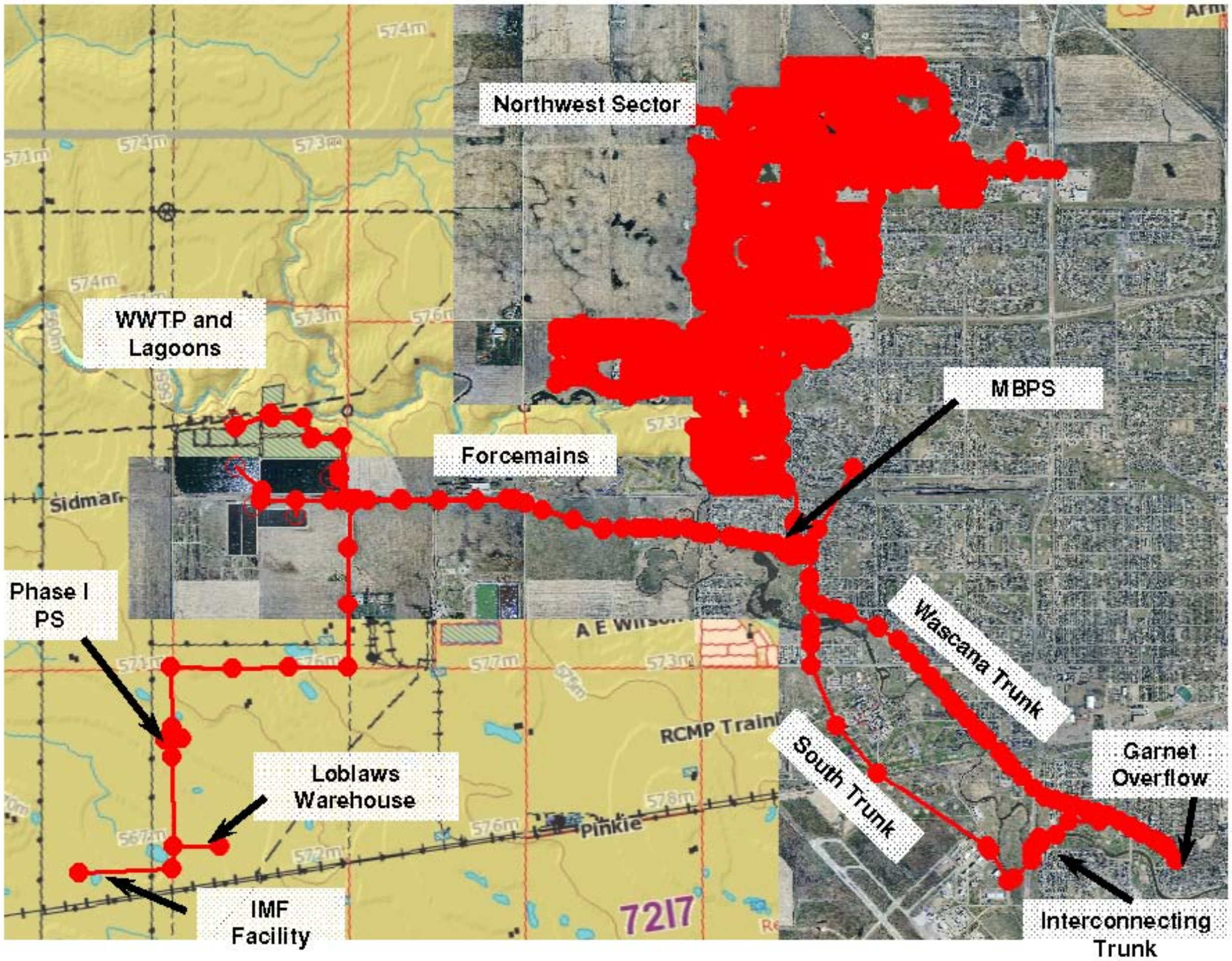
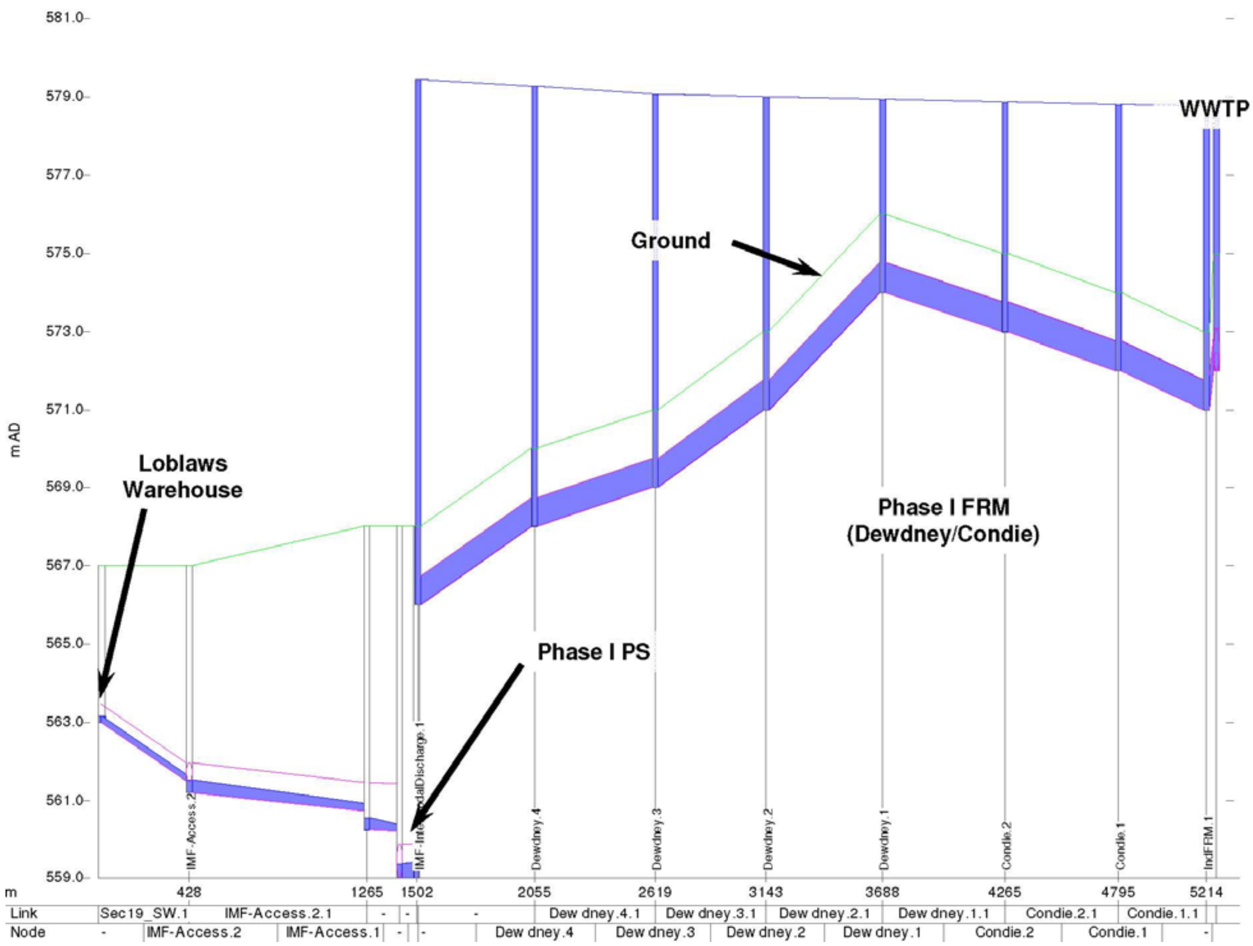


Table 11 – Summary of Total System Flows shows that the total system flows expected from Phase I Stage I of the proposed IMF will be approximately 10.5 MLD or roughly 120 L/s. Figure 15 – Hydraulic Grade Line Profile of Phase I - Stage I of IMF Facility Wastewater Collection System shows the expected hydraulic grade line profile for the wastewater collection system under the Stage I development scenario. The profile starts at the Loblaws node shown in Figure 14 and ends at the junction of the proposed IMF Facility forcemain and MBPS forceman.

Although not shown on the profile the maximum sewage flow rate into the Phase I Pumping Station based on the dynamic simulation is approximately 100 L/s which compares closely to the static peak flow rate estimated for the Stage I development of 120 L/s (10.5 MLD). The maximum flow rate within the IMF forcemain ranges from 150 L/s to 200 L/s. Peak flow rate into the WWTP site with the IMF Stage I added to the system is estimated to be 3,420 L/s under a 25-year design storm condition or 295 MLD. This flow compares to an existing flow rate into the WWTP site of 287 MLD under a 25-year design storm condition provided in the MBPS 1958 Forcemain Pre-Design Report Study. Stage I of the Phase I development, therefore adds roughly 8 MLD to the WWTP facility based on a dynamic flow analysis.

The profile shown in Figure 15 ties in at the VC which is located at the southeast corner of the WWTP site or at the terminus of the MBPS forcemain. The outlet pressure head at this locaton is approximately 579 m, which is consistent with Figure 5 – Internal Pressure Along South Forcemain at a Flow Rate of 300 MLD which shows the hydraulic grade line of the MBPS South forcemain under a flow rate of 300 MLD to the WWTP.

Figure 15 – Hydraulic Grade Line Profile of Phase I - Stage I of IMF Wastewater Collection System



The SW Servicing Study previously indicated that the South, Interconnecting and Pasqua trunks begin to experience levels of surcharge that would backup local systems and likely result in basement flooding under a 10 and 25-year rainstorm event. Figure 6 – Hydraulic Profile of South Trunk for Existing System Under Existing Level of Development In Response to a 10-Year Event shows the hydraulic grade line profile of the existing south trunk under existing conditions without the IMF connected to the wastewater system. This condition is the existing baseline in the southwest quadrant interceptor system.

Figure 16 – Hydraulic Profile of South Trunk for Existing System with IMF Stage I Development In Response to a 10-Year Event shows the same profile with Phase I Stage I of the IMF added to the existing wastewater collection system. A quick review of the two profiles shows that they are essentially identical. This suggest that adding in the Phase I Stage I wastewater flows at the terminus of the MBPS forcemain has little or no effect on the existing SW Sector interceptor system under a 10-year design storm condition.

Figure 16 – Hydraulic Profile of South Trunk for Existing System with IMF Stage I Development In Response to a 10-Year Event

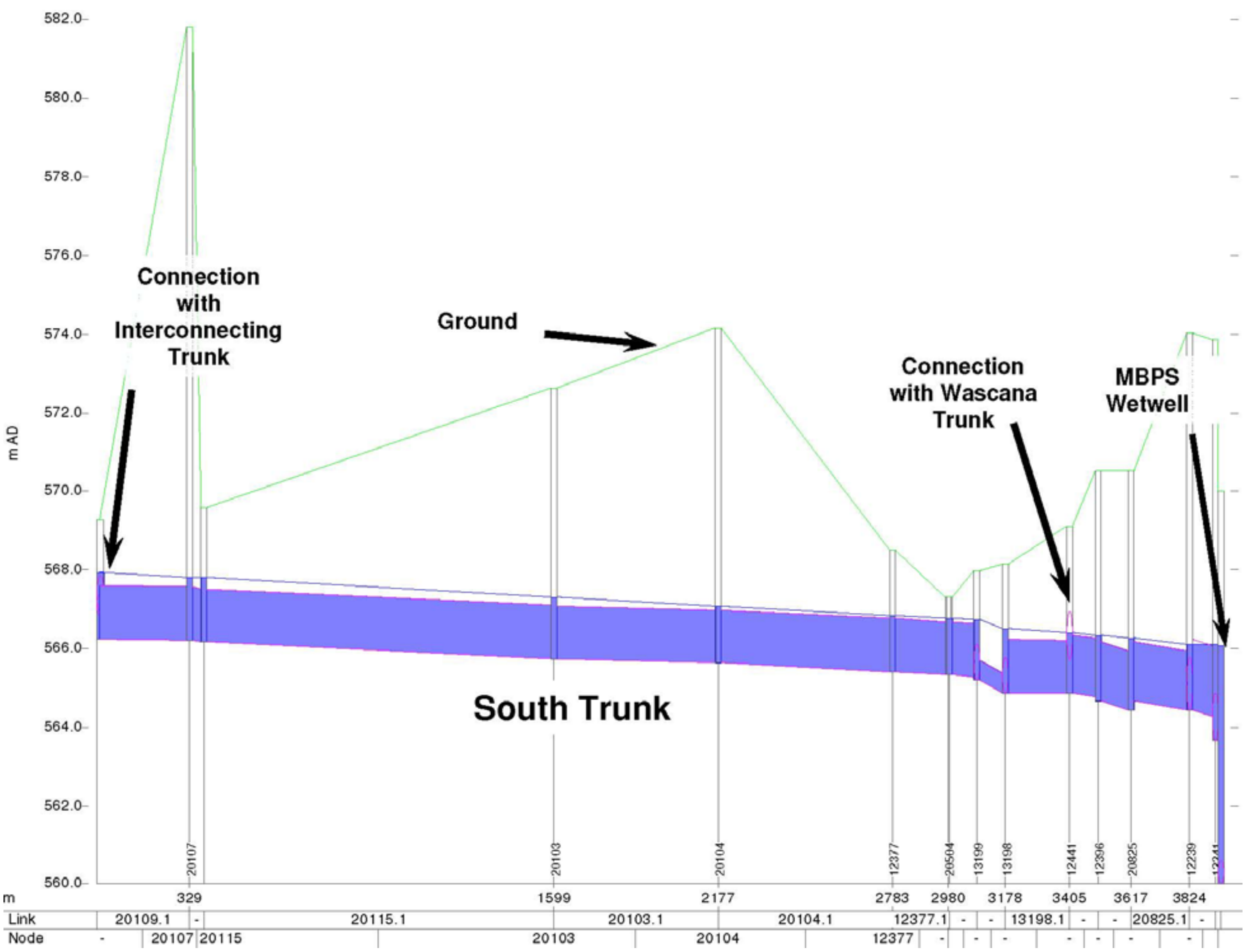
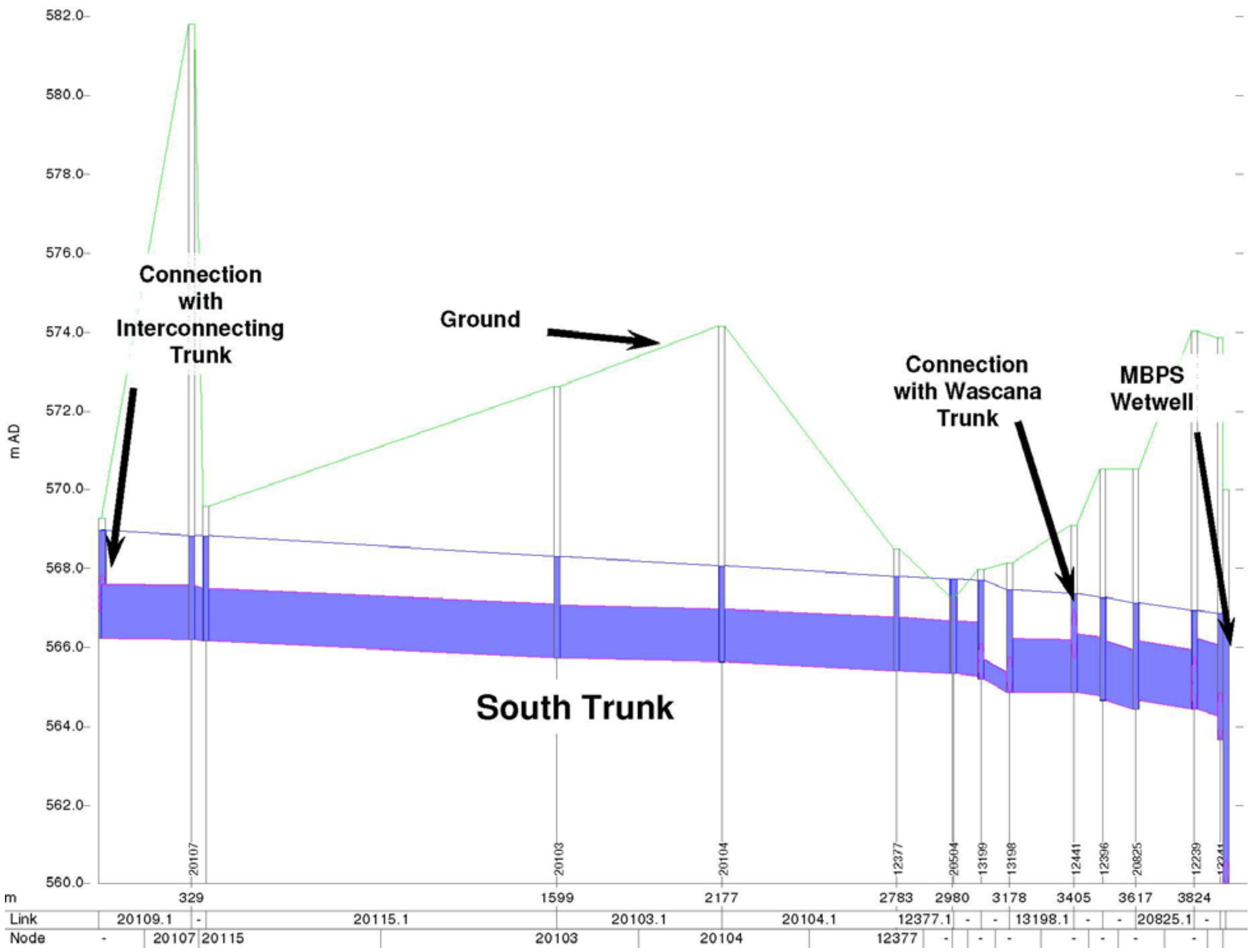


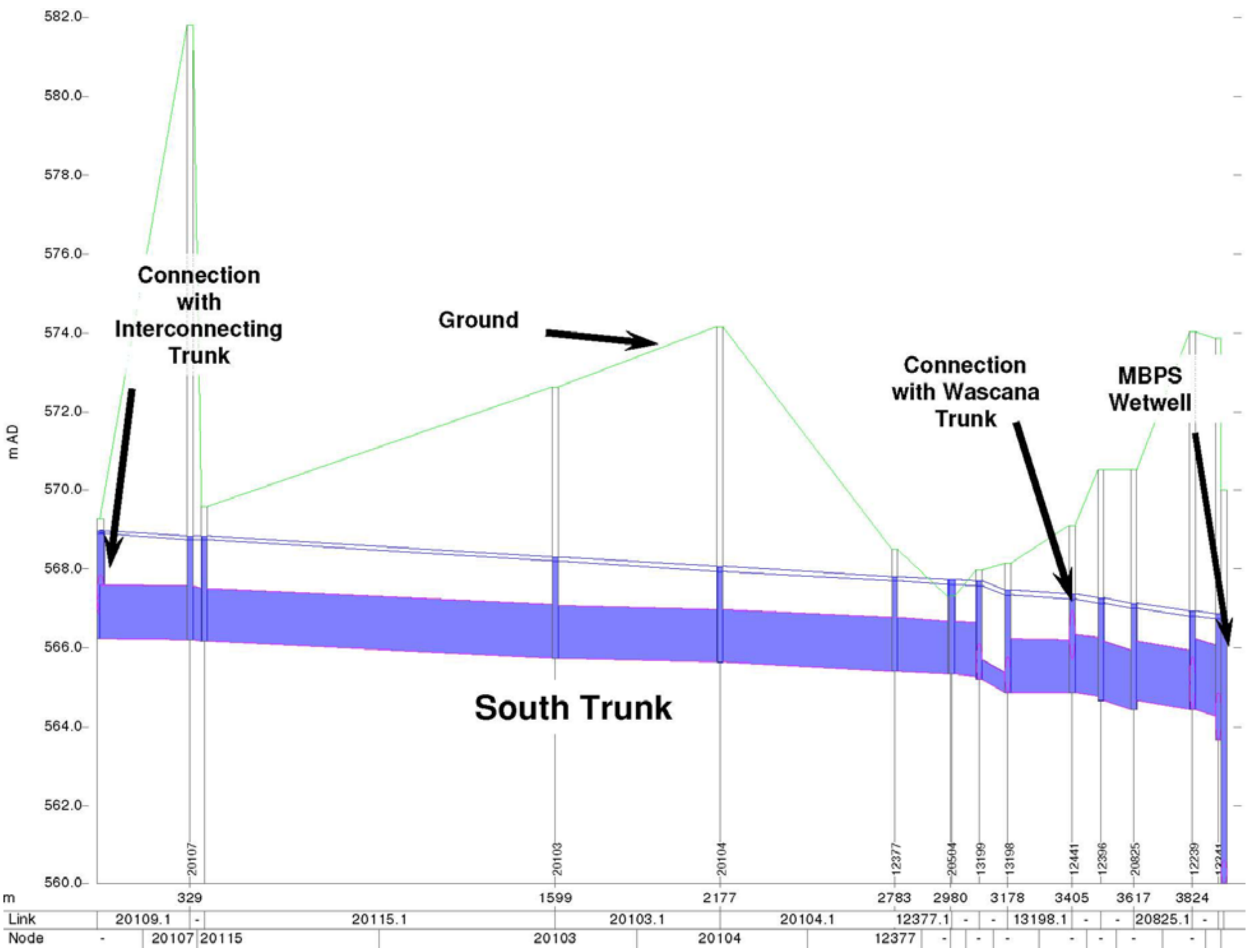
Figure 17 – Hydraulic Profile of South Trunk for Existing System With IMF Stage I Development In Response to a 25-Year Event shows the expected hydraulic grade line profile with Phase I Stage I of the IMF added to the existing wastewater collection system. This profile is again similar to Figure 7 which shows the existing hydraulic grade line profile under a 25-year storm event without the IMF added to the system.

Figure 17 – Hydraulic Profile of South Trunk for Existing System With IMF Stage I Development In Response to a 25-Year Event



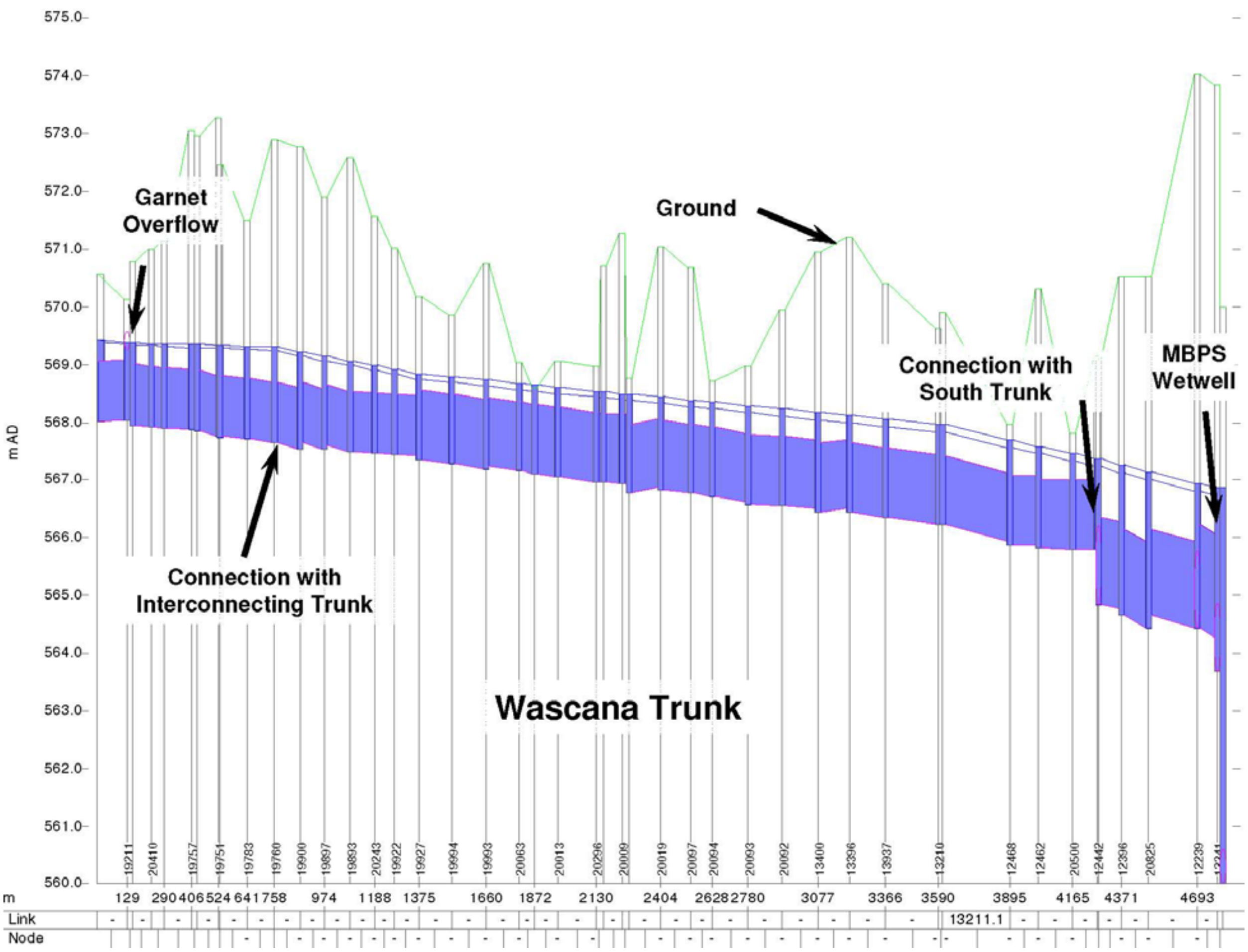
There is, however, a small difference as shown in Figure 18 – Hydraulic Grade Line Comparison of South Trunk With and Without Phase I Stage I Under Existing Conditions and a 25-Year Event, but the difference is again very small. The lower HGL is the HGL under existing conditions. The higher HGL is the HGL condition with Phase I Stage I of the IMF added to the existing wastewater collection system. For example, the HGL with Phase I Stage I added at the MBPS Wetwell is approximately 566.88 m versus 566.75 under existing conditions, or an increase of roughly 13 cm. The HGL at node 20107, near the upstream end, is 568.85 m with Phase I Stage I added versus 568.76 m under existing conditions, or an increase of 9 cm.

Figure 18 – Hydraulic Grade Line Comparison of South Trunk With and Without Phase I Stage I Under Existing Conditions and a 25-Year Event



Similarly Figure 19 – Hydraulic Grade Line Comparison of Wascana Trunk With and Without Phase I Stage I Under Existing Conditions and a 25-Year Event shows the difference in the HGL when the Phase I Stage I area is added to the existing wastewater collection system. The HGL at node 19211 will be approximately 569.38 m with Phase I Stage I added versus 569.35 m under existing conditions, or an increase of 3 cm at the top end of the Wascana Trunk.

Figure 19 – Hydraulic Grade Line Comparison of Wascana Trunk With and Without Phase I Stage I Under Existing Conditions and a 25-Year Event



The previous analysis shows that the Phase I Stage I development will have some impact on the existing wastewater infrastructure, but this impact will for the most part be isolated to the WWTP site which is already at or near capacity under existing loading conditions. The most important result of the analysis is that the proposed IMF Development will have some affect on the interceptor system in the SW quadrant of the City which is the area that is the most susceptible to basement flooding, but the affect will be minimal, i.e. the South Trunk closer to the upstream end will rise by roughly 9 cm compared to existing conditions while the Wascana Trunk near the Garnet Street Overflow will rise roughly 3 cm. It should be noted that the rise in the Wascana Trunk near its upstream end is held down to some extent since the overflow is operating under these conditions. Based on the analysis, the point at which overflows occur will remain relatively unchanged when compared to current conditions.

4.2.5.2 Expected System Response Phase I Stage III and Phase II of IMF Added

Table 11 – Summary of Total System Flows indicates that the total system flows expected from Phase I Stage III and Phase II of the proposed IMF and Industrial Lands Development will be approximately 69.5 MLD (45.8 MLD and 23.6 MLD from Phase I Stage III and Phase II, respectively) or roughly 800 L/s.

Figure 20 – Skeletal Model with IMF Phase I Stage III and Phase II Added shows the layout of the skeletal InfoWorks model developed to estimate the impact of the entire IMF and Industrial Lands Development on the existing wastewater system. See Drawing 06 – Wastewater Servicing – Servicing Review for the sizes of the assumed wastewater conduits and approximate pump station locations.

Figure 20 – Skeletal Model with IMF Phase I Stage III and Phase II Added

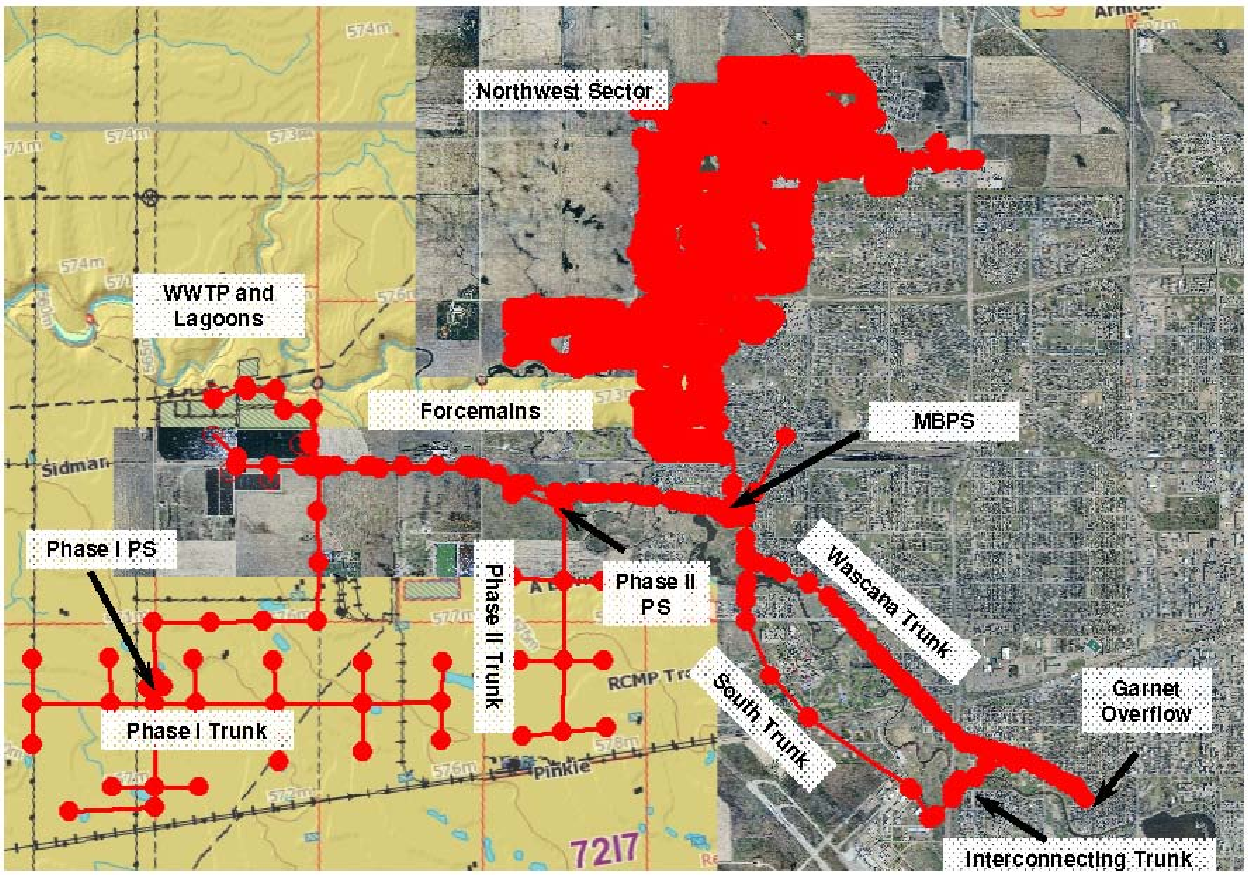


Figure 21 – Hydraulic Grade Line Profile of Phase I - Stage III of IMF Wastewater Collection System shows the expected hydraulic grade line profile for the wastewater collection system under the Stage III development scenario. The profile starts at the node located in the NE quadrant of Section 20 shown in Figure 20 and ends at the junction of the proposed IMF forcemain and MBPS forceman.

The maximum sewage flow rate into the proposed Phase I Pumping Station based on the simulation is approximately 450 L/s which compares closely to the static peak flow rate estimated for the Stage III development of 530 L/s (45.8 MLD). The maximum flow along the forcemain varies from 600 L/s to 520 L/s, (i.e. it varies based on the assumed pumping capacity of the IMF pumping station and its interaction with the Industrial Lands pumping station and the boundary conditions at the tie-in to the existing wastewater system). The maximum hydraulic grade line occurs immediately downstream of the pump station and is approximately 588.3 m. The pipe invert has been set at roughly 560 m indicating that the pressure head on the forcemain will be roughly 28 m or 40 psi. The boundary condition pressure at the VC is approximately 580.5 m.

Figure 21 – Hydraulic Grade Line Profile of Phase I - Stage III of IMF Wastewater Collection System

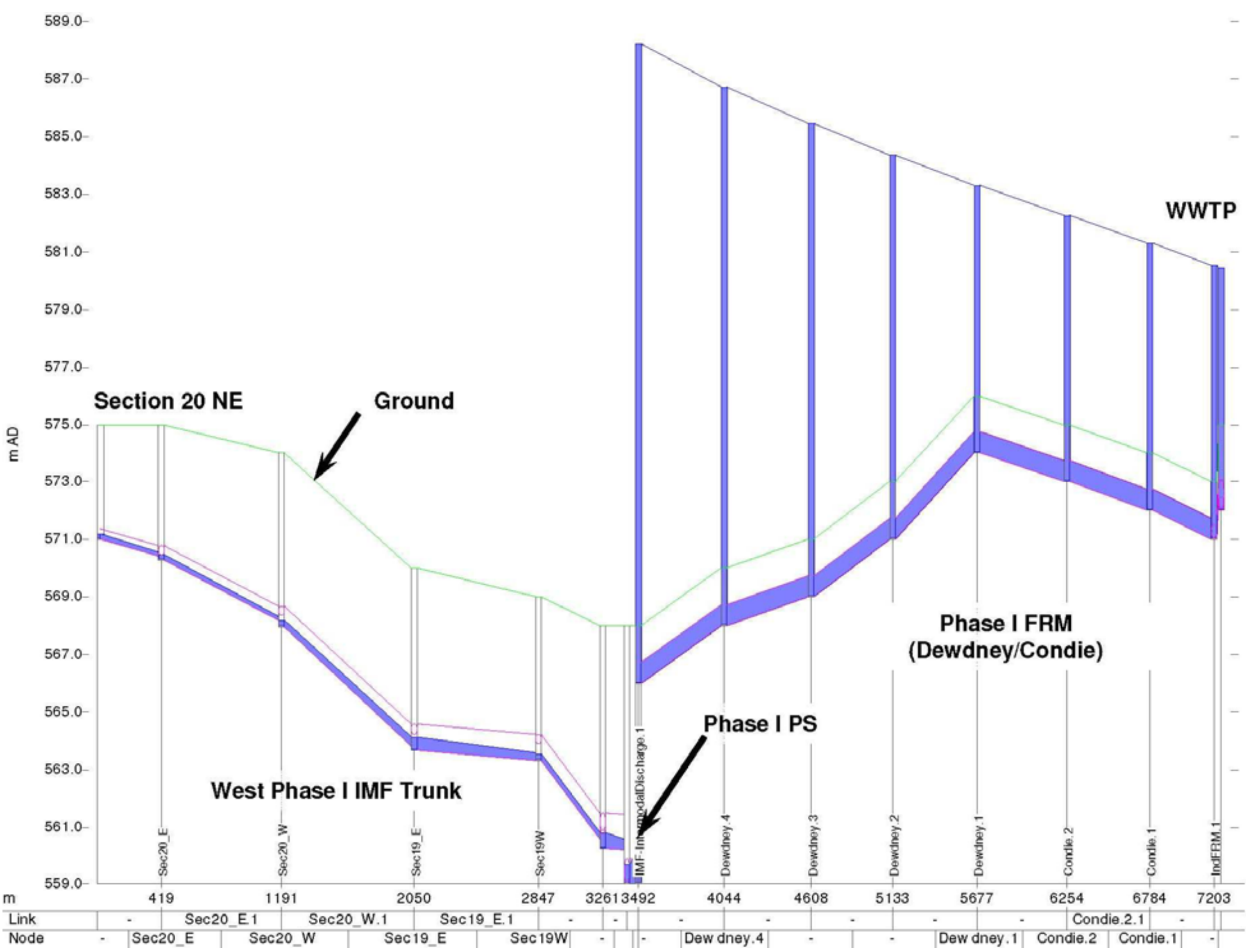
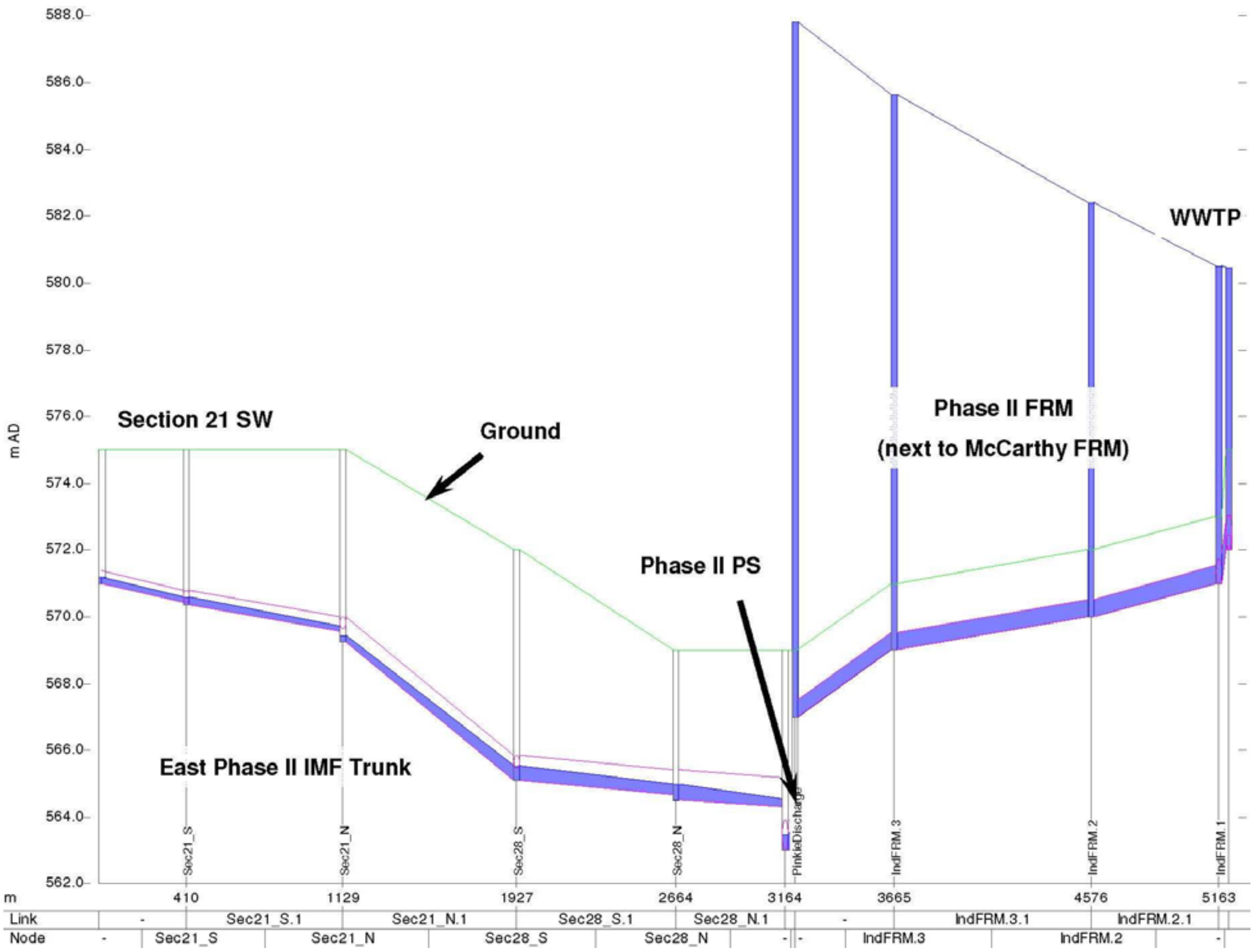


Figure 22 – Hydraulic Grade Line Profile of Phase II of the Industrial Lands Wastewater Collection System identifies the expected hydraulic grade line profile for the wastewater collection system for the Phase II development. The profile starts at the node located in the SW quadrant of Section 21 shown in Figure 20 and ends at the junction of the proposed Industrial Lands forcemain and MBPS forcemain.

The maximum sewage flow rate into the pumping station based on the simulation is approximately 250 L/s which compares closely to the static peak flow rate estimated for the Phase II development of 275 L/s (23.6 MLD). The maximum flow along the forcemain varies from 250 L/s to 300 L/s, i.e., it varies based on the assumed pumping capacity of the Industrial Lands pumping station and its interaction with the IMF pumping station and the boundary condition at the tie-in to the existing wastewater system. The maximum hydraulic grade line occurs immediately downstream of the pump station and is approximately 587.6 m. The pipe invert has been set at roughly 563.5 m indicating that the pressure head on the forcemain will be roughly 24 m or 35 psi. The boundary condition pressure at the VC is again approximately 580.5 m.

Figure 22 – Hydraulic Grade Line Profile of Phase II of the Industrial Lands Wastewater Collection System



Peak flow rate into the WWTP site with the IMF Phase I Stage III and Phase II added to the system is estimated to be 3,900 L/s under a 25-year design storm condition or 337 MLD. This compares to an existing flow rate into the WWTP site of 287 MLD under a 25-year design storm condition. The complete IMF and Industrial Land Development therefore adds roughly 50 MLD to the WWTP facility compared to existing conditions using a dynamic flow analysis.

Figure 23 – Hydraulic Grade Line Comparison of South Trunk With and Without Phase I Stage III and Phase II Under Existing Conditions and a 25-Year Event identifies the hydraulic grade line profile with the entire IMF and Industrial Land Development added to the existing wastewater collection system (higher profile). The existing 25-year HGL without the IMF and Industrial Lands Development is also shown on the profile (lower HGL).

Figure 23 – Hydraulic Grade Line Comparison of South Trunk With and Without Phase I Stage III and Phase II Under Existing Conditions and a 25-Year Event

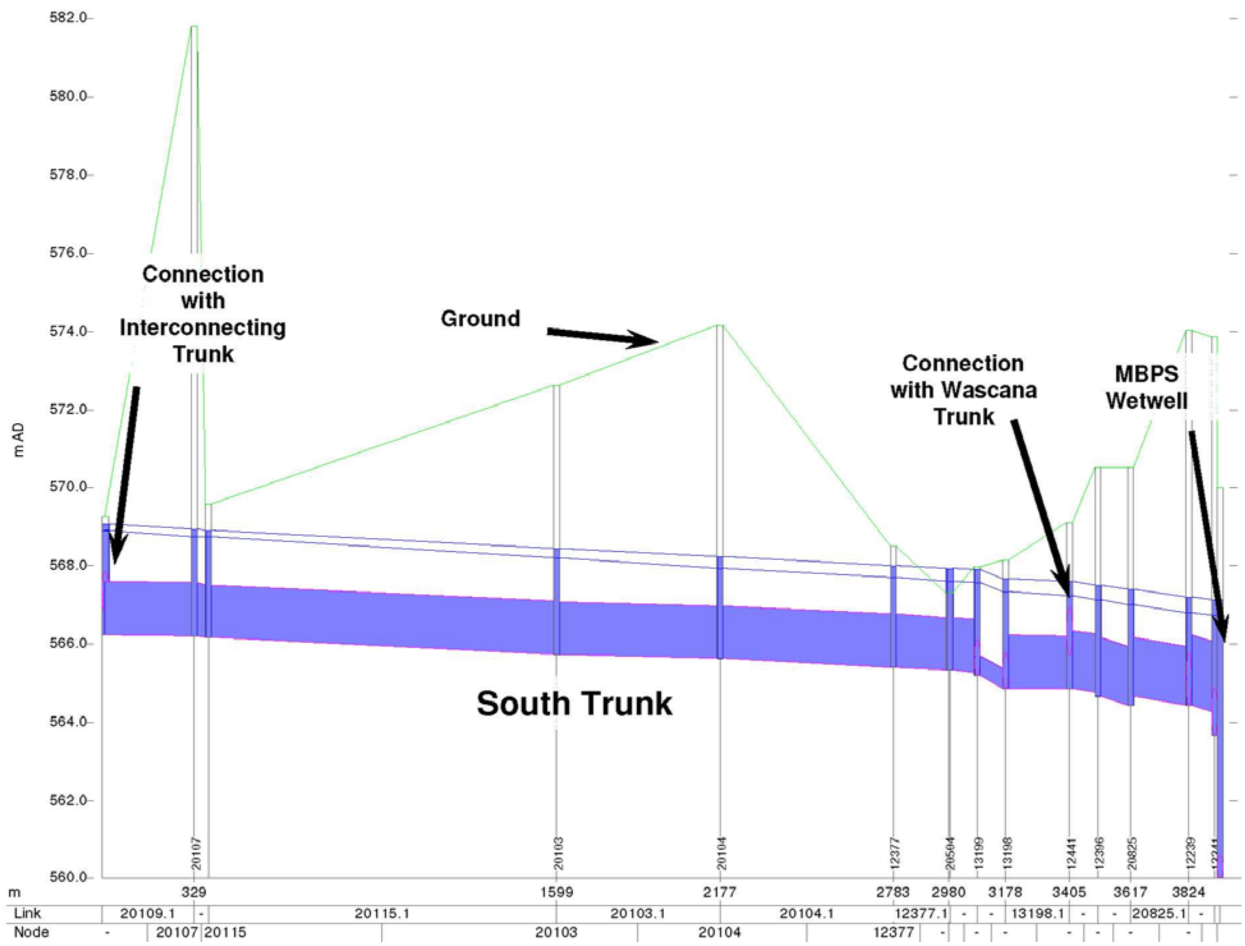
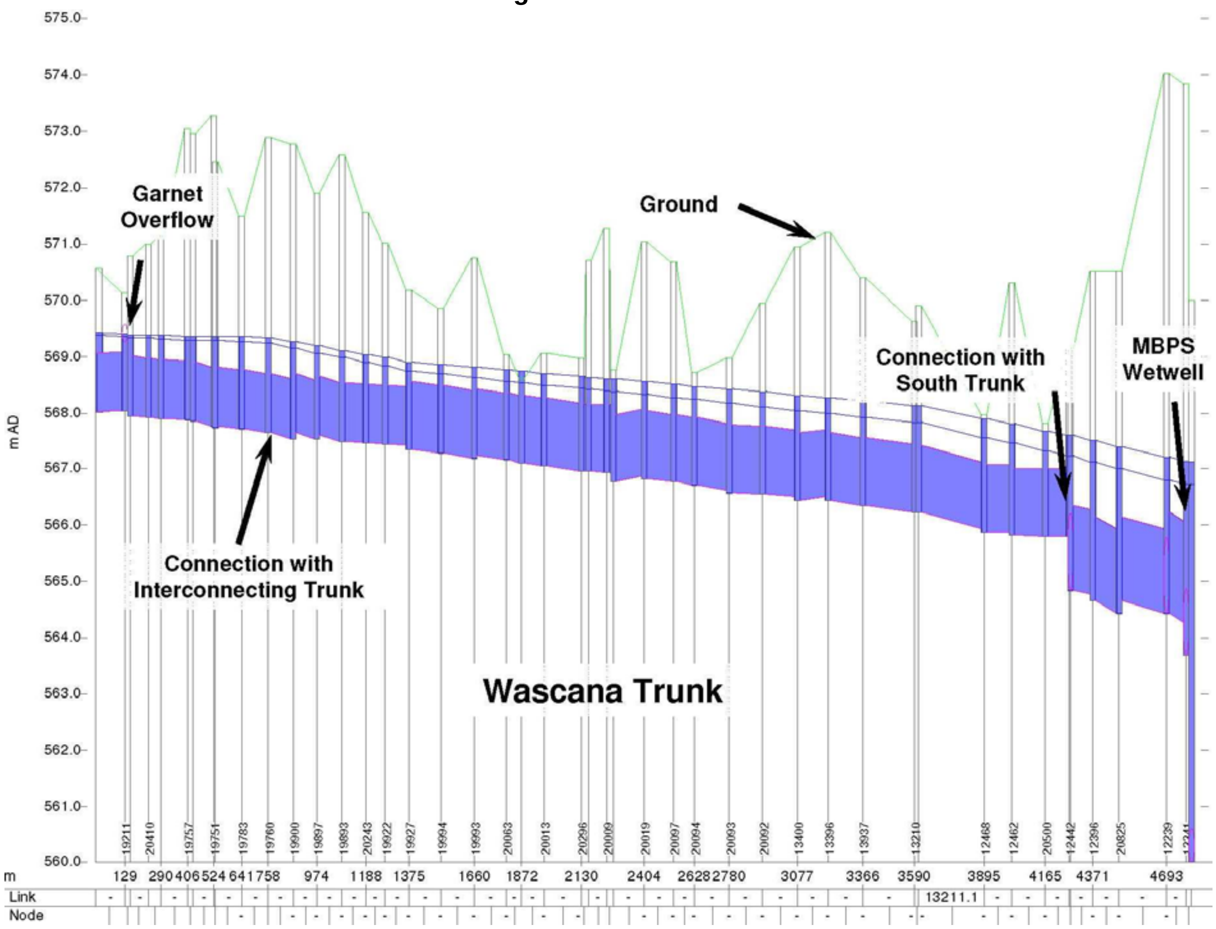


Figure 24 – Hydraulic Grade Line Comparison of Wascana Trunk With and Without Phase I Stage III and Phase II Under Existing Conditions and a 25-Year Event shows a similar increase in the HGL levels during a 25-year rainstorm event. Under these conditions, the HGL's at the lower end are increased by roughly 0.25 m. The increase is not as pronounced at the upstream end because of the Garnet Street Overflow that starts to overflow into the Wascana Creek at an elevation of 569.3 m.

Figure 24 – Hydraulic Grade Line Comparison of Wascana Trunk With and Without Phase I Stage III and Phase II Under Existing Conditions and a 25-Year Event



Previous analysis suggests that adding Phase I Stage III and Phase II development into the existing wastewater collection system will impact the SW quadrant interceptor system which could produce additional basement flooding in the area under severe rainstorm events. Some form of relief of the interceptor system such as the twinning of the South Trunk mentioned in Section 4.2.4.2 – Southwest Sector Wastewater Servicing Analysis will likely be required under these loading conditions in order to mitigate the effects of adding the IMF Development to the wastewater collection system.

The other major impact will be the increased flows to the WWTP which are expected to increase by roughly 50 MLD under a 25-year design storm condition, i.e. from 287 MLD to 337 MLD. Existing plant capacity is rated to be approximately 250 MLD which implies the current capacity is not adequate, even though the plant is capable of receiving up to 300 MLD for short periods of time, according to the MBPS 1958 Forcemain Pre-Design Report Study.

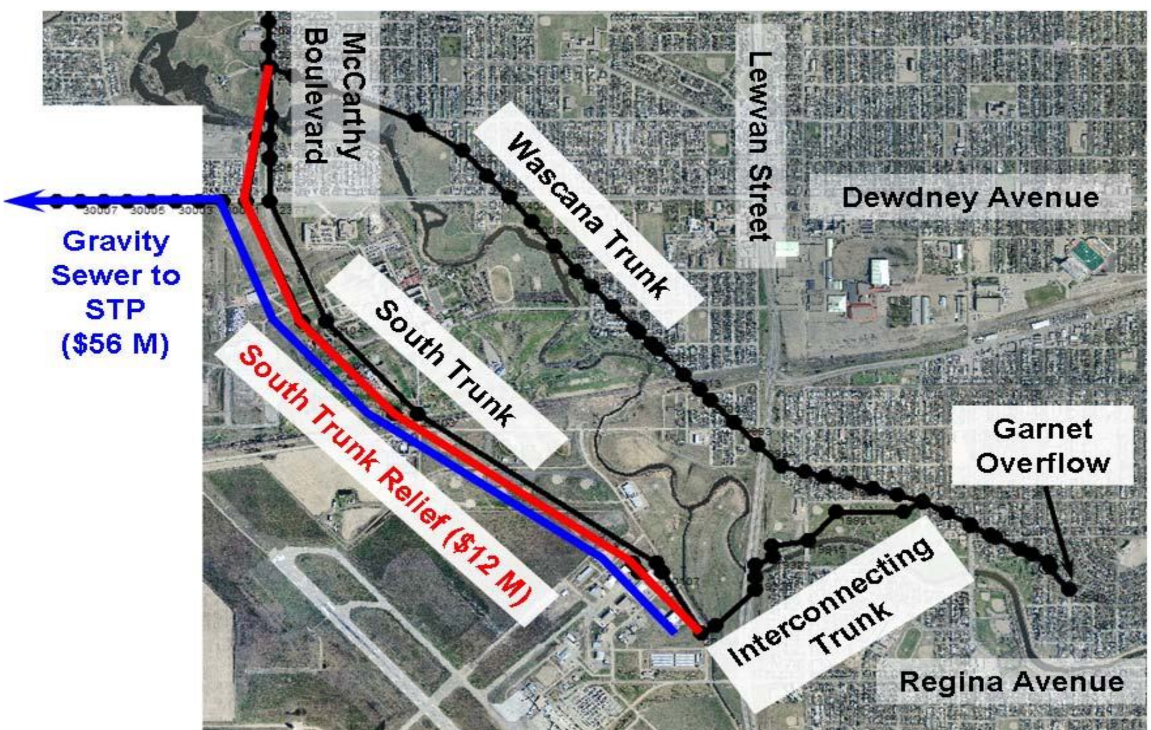
4.2.5.3 Expected System Response to 235,000 Target-Population Conditions

The inflows associated with the future levels of development for the SW and NW sectors were examined as part of this study in order to estimate the expected system response of the future wastewater collection system to the IMF and Industrial Lands Development. The Stage I development modeled in the NW Sector Study and the Stage I and Stage II development modelled in the SW Servicing Study were added to the existing wastewater collection system to simulate the future loading conditions.

These stages of development in the southwest and northwest quadrants represent an estimated population growth of approximately 35,000 residents, or nearly all the growth necessary to get to the 235,000 target-population level in the City of Regina. Based on growth rates of between 0.5% and 1%, the 235,000 target-population would be achieved in approximately 20 to 40 years, which is the roughly equivalent to the time estimated to complete the IMF and Industrial Lands Development.

The SW Servicing Study indicated that the South, Interconnecting and Pasqua trunks experience levels of surcharge that would backup local systems and likely result in basement flooding under extreme rainfall events. Preventing surcharge of the South Trunk would, therefore, help minimize the risk of basement flooding. The relief concept recommended in the MBPS 1958 Forcemain Pre-Design Report consists of twinning 3,400 m of the South Trunk from the Interconnecting Trunk to the Wascana Trunk at a cost of roughly $12M based on 2006 construction values. The relief concept is shown in red in Figure 25 – Relief Options for South Trunk below. The InfoWorks model used to examine the impacts of the IMF and Industrial Lands Development on the future wastewater system was configured assuming the twinned South Trunk option, i.e., 3,400 m of conduit, was added to the model adjacent to the existing South Trunk.

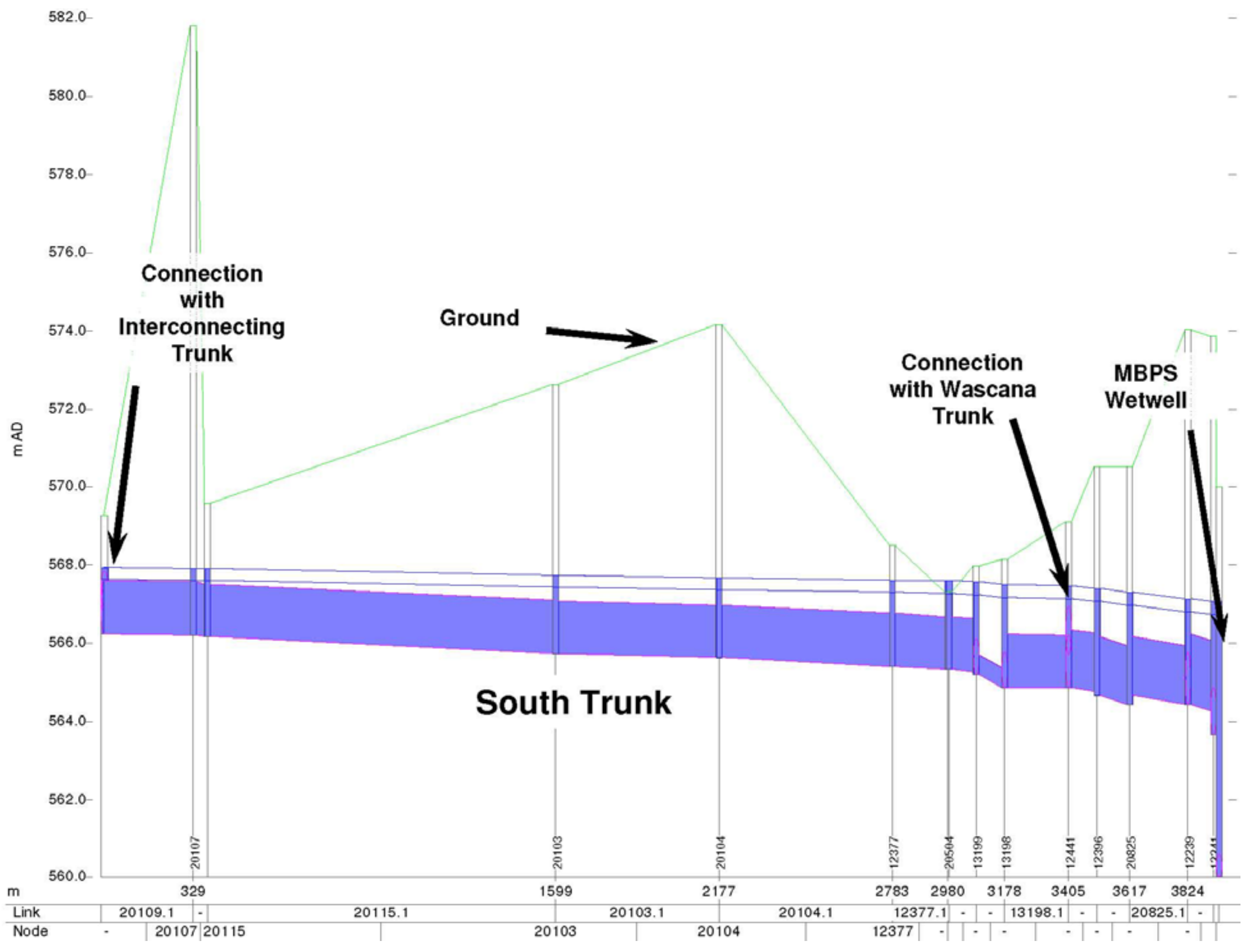
Figure 25 – Relief Options for South Trunk



Peak flow rate into the WWTP site with the IMF Phase I Stage III and Phase II added to the future wastewater collection system will be 3,900 L/s under a 25-year design storm condition or 337 MLD. This flow rate is the same as the previous analysis under the existing wastewater collection system with the IMF and Industrials Land Development added to the system.

Figure 26 – Hydraulic Grade Line Comparison of South Trunk With and Without Phase I Stage III and Phase II Under Future Conditions and a 25-Year Event depicts the hydraulic grade line profile with the entire IMF and Industrial Land Development added to the future wastewater collection system (higher profile). This configuration includes twinning of the south trunk. The expected future 25-year HGL without the IMF and Industrial Lands Development is also shown on the profile (lower HGL).

Figure 26 – Hydraulic Grade Line Comparison of South Trunk With and Without Phase I Stage III and Phase II Under Future Conditions and a 25-Year Event

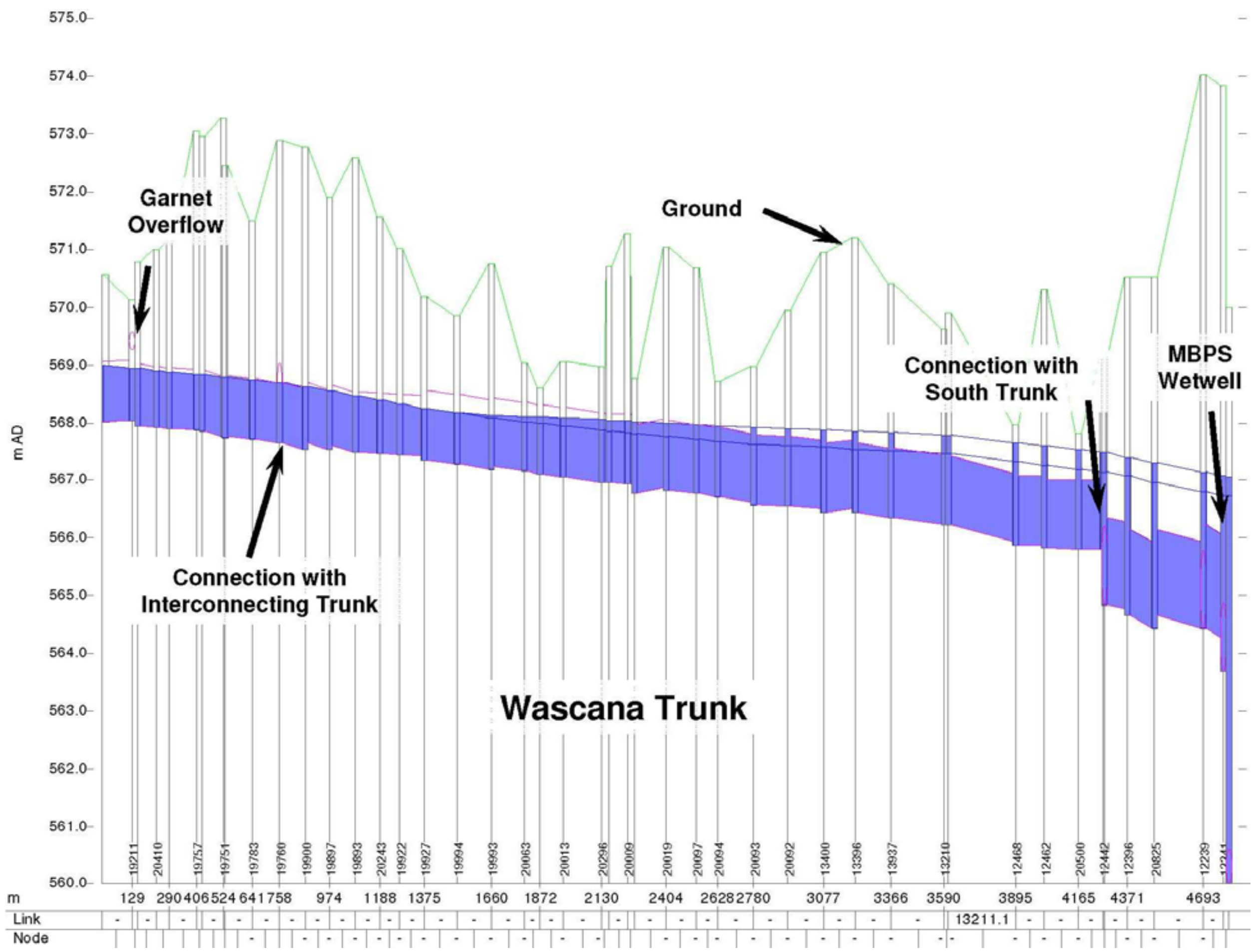


The grade line shows that the IMF and Industrial Lands Development will have an impact on the South Trunk HGL for the future wastewater collection system even with implementation of the recommended South Trunk twinning relief option. The lower HGL represents the expected hydraulic performance of the future wastewater collection system with Stage 1 and Stage 2 developments identified in the SW Sector

Servicing Study added to the SW Sector quadrant interceptor system and the South Trunk twinned as previously discussed. The upper HGL represents the changed conditions with the IMF and Industrial Lands Development added to the system.

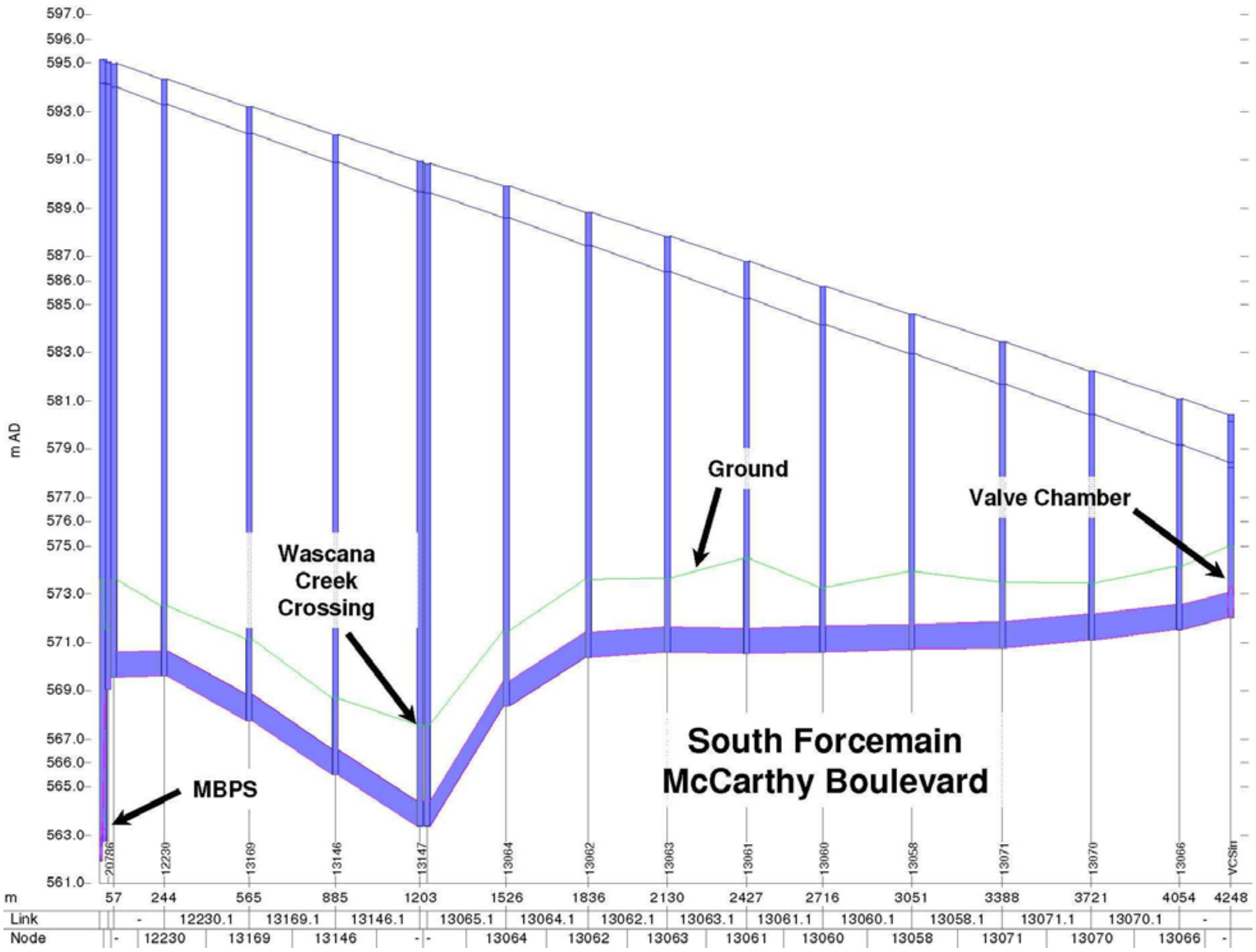
Figure 27 – Hydraulic Grade Line Comparison of Wascana Trunk With and Without Phase I Stage III and Phase II Under Future Conditions and a 25-Year Event shows a similar comparison of the HGL's with and without the IMF and Industrial Lands Development added to the system along the Wascana Trunk.

Figure 27 – Hydraulic Grade Line Comparison of Wascana Trunk With and Without Phase I Stage III and Phase II Under Future Conditions and a 25-Year Event



The analysis above suggests that the IMF and Industrial Lands Development will have an impact on the SW Sector interceptor system even with the future recommended relief envisioned for the South Trunk added to the system. The reason that the IMF and Industrial Lands Development impacts the SW interceptor system to the degree that it does appears to be due to the increase in the HGL of the MBPS forcemain as shown in Figure 28 – Hydraulic Grade Line Comparison of South Forcemain With and Without Phase I Stage III and Phase II Under Future Conditions and a 25-Year Event.

Figure 28 – Hydraulic Grade Line Comparison of South Forcemain With and Without Phase I Stage III and Phase II Under Future Conditions and a 25-Year Event



The top HGL represents the peak forcemain pressures with the IMF and Industrial Land Development added to the future wastewater system. The HGL at the terminus of the forcemain under these conditions is approximately 580.5 m versus 578.5 m without the development flows, ie. an increase of 2 m. The HGL at the upstream end of the forcemain increases from 594.2 to 595.1 m; ie. an increase of approximately 0.9 m. This is then reflected back into the interceptor system. One way to alleviate this situation is to reduce the HGL at the downstream end of the MBPS forcemain. This reduction could be achieved by twinning the existing 1350 mm conduit leading from the VC to the PTP as suggested in the MBPS 1958 Forcemain Pre-Design Report.

4.2.6 Wastewater System Cost Estimates

Table 15 – Conceptual Wastewater System Cost Estimate below summarizes the estimated costs of the regional services required for the various stages and phases of development as described in Section 2 and shown on Drawings 08 through Drawing 11.

Table 15 – Conceptual Wastewater System Cost Estimate

Phase	Stage	Description	Cost	Total
I	I	Wastewater Sewers	3,850,000	
		Forcemain	10,800,000	
		Pump Station	9,960,000	
			Total for Stage I	$ 24,610,000
I	II	Wastewater Sewers	16,500,000	
		Forcemain	500,000	
		Pump Station	0	
			Total for Stage II	$ 17,000,000
I	III	Wastewater Sewers	3,850,000	
		Forcemain	150,000	
		Pump Station	0	
			Total for Stage III	$ 4,000,000
			Total for Phase I	**$ 45,610,000**
II		Wastewater Sewers	8,900,000	
		Forcemain	3,650,000	
		Pump Station	6,800,000	
			Total for Phase II	**$ 19,350,000**
			Total for Phase I & II	**$ 64,960,000**

As noted in the table a considerable amount of upfront infrastructure is required to service the Phase I Stage I development due to the relatively remote location of the proposed IMF.

4.2.7 Wastewater Servicing Summary

A wastewater servicing analysis was conducted for the proposed IMF and Industrial Lands development using the City of Regina Development Standards as a basis for design. Based on the proposed phasing and staging of the development and the existing topography in the area, the proposed servicing was divided into two discrete wastewater systems. One system services the IMF and the lands located west of Pinkie Road while the other services the areas east of Pinkie Road. The proposed wastewater servicing systems consist of gravity sewers and pumped forcemain systems. These forcemains connect at the terminus of the existing McCarthy Booster Pumping Station forcemains which tie into the existing WWTP at the intersection of 1st Avenue North and Condie Road.

A dynamic flow analysis using the existing City of Regina interceptor model was conducted to determine the possible impact of the proposed development on the existing wastewater infrastructure. It was shown that the Phase I Stage I level of development will not create a significant impact on the existing wastewater infrastructure. Further developments, i.e., Phase I Stage II and Stage III and Phase II will impact both the existing WWTP and the interceptor sewer system located in the SW Sector of Regina. Additional upgrades to the existing WWTP and interceptor system in the SW Sector will be required if the additional stages and phases of development proceed.

The estimated total cost of the first stage, i.e. Phase I Stage I, is expected to be approximately $25M. The total development wastewater servicing cost for Phase I & II is expected to be reach approximately $65M.

4.3 Water Servicing

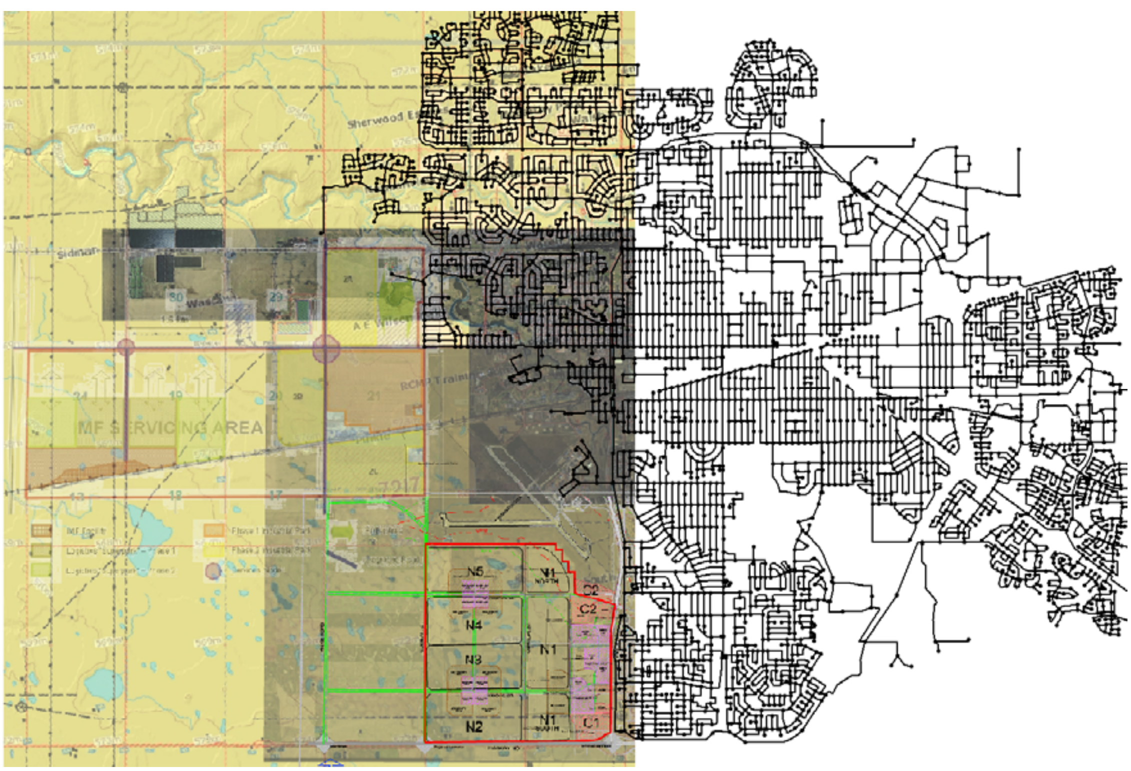
The primary purpose of the water servicing review was to:

- Describe the existing water distribution system in the vicinity of the proposed development and identify any major constraints
- Estimate additional demands that will be generated by the proposed development
- Estimate the expected water distribution system response to the additional demands and fire flows
- Recommend short term and long term infrastructure improvements to allow the proposed development to proceed
- Identify the financial implications associated with providing this servicing and/or improvements to the water system.

4.3.1 Existing Water System and Major Capacity Constraints

Similar to the wastewater system servicing analysis, very little water infrastructure exists in the vicinity of the proposed development as shown in the figure below.

Figure 29 – Regina's Water Distribution System



Furthermore, the water infrastructure that does exist near the western edge of the City limits has limited capacity, particularly for the areas located southwest of Wascana Creek. This area includes the RCMP Training Facility.

Two servicing alternatives were reviewed as part the conceptual water servicing analysis. The first alternative was comprised of a conventional feed from the existing water distribution system. Due to the limited capacity of the existing water distribution system near the western fringes of the City, this feed had to be tied back farther into the system so that it could provide the necessary water supply and fire flows to the IMF and Loblaw's warehouse. The other alternative that was reviewed was a local reservoir with a smaller feed connected to the existing water distribution system near the western limits of the system. This reservoir and accompanying pump station would need to be sized to provide the necessary fire flows and water storage for the IMF and Lowblaw's warehouse facility.

The first alternative requires a large 600 mm pipeline that connects back into the 875 mm or 1050 mm inner feeder main loop located on Lewvan Drive. The closest connection point is at Lewvan Drive and 13th Avenue as shown in Figure 30 – Alternative 1 – 600 mm Feed From Lewvan Drive and 13th Avenue. The proposed feed would follow west along the 13th Avenue right-of-way up to Courtney Road. It would then turn north on Courtney Road and deflect west again along Dewdney Avenue before turning south towards the IMF as shown on the figure.

Figure 30 – Alternative 1 – 600 mm Feed From Lewvan Drive and 13th Avenue

The second alternative would be fed from the nearest existing 300 mm waterline which is located at the intersection of Read Avenue and Toothill Street as shown in Figure 31 – Alternative 2 – Reservoir at Dewdney Avenue and IMF Access. This second alternative, however, was ultimately rejected because it was determined that the existing water distribution system near the western limits of the City could not provide the necessary maximum day water demands which would be the minimum requirement for a reservoir alternative. It was also not amendable to staged development and would be difficult to operate and maintain over the course of the IMF and Industrial Lands development.

Figure 31 – Alternative 2 – Reservoir at Dewdney Avenue and IMF Access (Obsolete - Rejected)

4.3.2 Existing information

4.3.2.1 Topographic Information

Topographic information was obtained from the 10 metre National Topographic Survey (NTS) maps and the Government of Canada Toporama website, which generates roughly 1 metre contours based on the NTS 10 metre contours. This contour information was used to supplement the information available in the existing City of Regina water distribution model.

4.3.2.2 Hydraulic Model

The City of Regina maintains a hydraulic model of the existing water distribution system. The extent and limits of this model are shown in Figure 29 – Regina's Water Distribution System which shows the entire conduit links currently configured in the model. The City's model is maintained in the WaterCAD software platform developed by Haestad Methods. This model was converted to the EPANET simulation software package for the water analysis. EPANET is an open-source water distribution simulation software package developed and maintained by the U.S. EPA. EPANET is also the computational engine used by WaterCAD software, so the results obtained should be identical to the original WaterCAD model.

The City's water model is large, but relatively straightforward since no pumping or complex controls have been directly simulated in the model to date. For example, the North Pumping Station and the Farrell Pumping Stations, which are the only sources of supply included in the model at this time, are modeled as constant head reservoirs, i.e., sources that have infinite storage and pumping capacity.

4.3.3 Design Parameters

Initial water demands for the proposed development were developed based on the City of Regina's Development Standards Manual[8]. Table 16 – Water Main Model Parameters and Table 17 – Water Main Model Conditions. are a summary of the design parameters used in the water servicing review.

Table 16 – Water Main Model Parameters

PARAMETER	VALUE
Average Day Consumption	415 LPCD
Maximum Day Peaking Factor	2.1 x Average Day
Maximum Hour Peaking Factor	3.2 x Average Day
Hazen Williams Coefficient (HDPE)	130
Dry Industrial Design Population	25 persons/Ha
Mix Industrial Design Population	35 persons/Ha
Minimum Fire Flow Industrial Buildings	158 Lps (2500 USgpm)

Table 17 – Water Main Model Conditions.

CONDITIONS	CONSTRAINT
Max Water Main Velocity (under fire flow conditions)	<= 3.2 m/s
Minimum Fire Flow + Max Day residual pressures	140 kPa
Peak Day Demand Pressure Drop	<= 17kPa
Peak Day Demand Minimum Pressure at Water Main	>= 280 kPa
Peak Hour Demand Pressure Drop	<= 27.6kPa
Peak Hour Demand Minimum Pressure at Water Main	>= 27.6 kPa
Peak Hour Maximum Design Velocity	<= 1.5 m/s
Peak Day + Fire Flow Maximum Design Velocity	<= 3.2 m/s

4.3.4 Water Demand Estimates

Equivalent design populations were calculated in accordance with City of Regina Development Standards. They are summarized in Table 18 – Equivalent Population Estimates.

[8] *City of Regina, "Development Standards Manual, Section 8.0 Domestic Sewer Design Standards"*

Table 18 – Equivalent Population Estimates

PHASE I					
Node	Land Use	Total Area (Ha)	Building Area (Ha)	Population Density (persons/Ha)	Total Population
Phase I - Stage I (Short Term 0 - 2 years)					
Intermodal Facility CPR					
N Section 13	Industrial	79	4.0	25	100
NW Section 18	Industrial	51	2.5	25	63
Loblaw					
SW Section 19	Dry Industrial	64	48.0	25	1,200
Subtotal Stage I		194	54.5		1,363
Phase I - Stage II (Medium Term 2 - 25 years)					
Dry Industry Area West Pinkie Road					
SE Section 24	Dry Industrial	64	48.0	25	1,200
SW Section 24	Dry Industrial	64	48.0	25	1,200
NE Section 18	Dry Industrial	17	12.5	25	313
SE Section 19	Dry Industrial	64	48.0	25	1,200
Mixed Industrial Area					
NW Section 20	Mixed Industrial	64	48.0	35	1,680
SW Section 20	Mixed Industrial	64	48.0	35	1,680
NW Section 17	Mixed Industrial	7	5.3	35	186
E Section 20 – (Sakimay)	Mixed Industrial	115	86.3	35	3,021
Subtotal Stage II		459	344.1		10,480
Phase I - Stage III (Long Term beyond 25 years, above 235,000 population)					
Dry Industry Area West Pinkie Road					
NW Section 19	Dry Industrial	64	48.0	25	1,200
NE Section 19	Dry Industrial	64	48.0	25	1,200
NW Section 24	Dry Industrial	64	48.0	25	1,200
NE Section 24	Dry Industrial	64	48.0	25	1,200
Subtotal Stage III		256	192.0		4,800
Total Phase I		909	590.6		16,643
PHASE II					
Node	Land Use	Total Area (Ha)	Building Area (Ha)	Population Building (Ha)	Total Population
Mixed Industrial Area East Pinkie Hwy (Long Term beyond 25 years, above 235,000 population)					
Section 28	Mixed Industrial	165	123.8	35	4,333
Section 21	Mixed Industrial	200	150.0	35	5,250
Total Stage I		365	273.8		9,583
Total Annexation area		1,274	864.4		26,226

Table 19 – Water Demand Calculations shows a summary of the water demands calculated for the various phases and stages of development as described in Section 2 of this report. The table shows that the total maximum day and peak hour demands for the Phase I Stage I are estimated to be a modest 13.9 L/s and 21.2 L/s, respectively. Stage II on the other hand jumps to values of roughly 120 L/s and 182.9 L/s. Similarly for Stage III the total maximum day water demands increase to 168.9 L/s while the maximum hour demands increase to 257.3 L/s. The total demands for Phase I and Phase II fully developed are estimated to be 265.5 and 404.5 L/s, respectively. These demands cannot be accommodated by the existing water distribution system located near the western limits.

Table 19 – Water Demand Calculations

PHASE I - Short Term (0-25 years)							
Node	**Total Gross Area (Ha)**	**Develope d Area (Ha)**	**Population Density per (person Ha)**	**Total Population**	**Total Avg Day Demand (LPS)**	**Max Day (LPS)**	**Max Hour (LPS)**
Phase I - Stage I (Short Term 0 – 3 years)							
Sub total Stage I	**194**	**54.5**	**25**	**1,363**	**6.6**	**13.9**	**21.2**
Phase I - Stage II (Medium Term 3 - 25 years)							
Dry Light Industrial - Area West Pinkie Hwy	209						
Mixed Light Industrial - Area East Pinkie Hwy	250						
Sub total Stage II	**459**	**344.1**	**25/35**	**10,480**	**50.5**	**106.2**	**161.7**
Phase I - Stage III (Long Term beyond 25 years, above 235,000 population)							
Dry Light Industrial - Area West Pinkie Hwy							
Sub total Stage III	**256**	**192**	**25**	**4,800**	**23.2**	**48.8**	**74.4**
Total Phase I	**909**	**590.6**		**16,643**	**80.3**	**168.9**	**257.3**
PHASE II - Long Term (Beyond 25 Years)							
Mixed Light Industrial - Area East Pinkie Hwy (Long Term beyond 25 years, above 235,000 population)							
Total Phase II	**365**	**273.8**	**35**	**9,583**	**46.00**	**96.60**	**147.20**
Total Annexation area	**1274**	**864.4**		**26,226**	**126.3**	**265.5**	**404.5**

4.3.5 Conceptual Water Analysis

In order to be consistent with other recent planning and servicing analysis such as the recently completed SW and NW Sector servicing studies, the water distribution model (EPANET) was configured to approximately mimic a target population of 235,000 people. This configuration was done by adjusting the demand multiplier parameter in the EPANET model to produce the following demand flows from the two existing reservoirs:

- Total maximum day demand = 205 MLD.
- Total peak hour demand = 312 MLD.

These demands are based on a population of 235,000 people that consume 415 lpcd using maximum day and maximum hour peaking factors of 2.1 and 3.2, respectively.

The following is a brief description of the upgrades to the proposed distribution system for the various phases and stages of the development.

4.3.5.1 Phase I

Stage I - Short Term (0 – 3 years)

- The proposed water distribution system to supply the IMF and Loblaw's warehouse sites, i.e., Stage I in Phase I, was sized to provide a domestic flow capacity of approximately 13.9 L/s (maximum day demand for 54.5 Ha of Industrial Development -195 Ha gross area) plus a fire flow capacity of 158 L/s (2500 USgpm) at a minimum pressure of 140 kPa.

The IMF and adjacent development will be served by a 600 mm feed, connected to the 857 mm feedermain City Loop at Lewvan Drive. The 600 mm main runs west on 13th Avenue to Courtney Street, north on Courtney Street to Dewdney Avenue and west on Dewdney Avenue to the IMF access road to the IMF and south on the access road to provide domestic and fire flows to the IMF site and Loblaw's local distribution systems. See Figure 30 – Alternative 1 - 600 mm Feed From Lewvan Drive and 13th Avenue and Drawing 08 – Storm Water – Wastewater – Water – Phase I - Stage I.

Stage II - Medium Term (3 – 25 years)

- The proposed water distribution system for the Mixed and Dry Light Industrial Development included in Stage II Phase I was upgraded to accommodate an additional domestic maximum day flow capacity of 106.2 L/s i.e. maximum day demand for 344 Ha (460 Ha gross area) plus the maximum day flows required for Stage I of 13.9 L/s plus the fire flow requirement of 158 L/s.
- A number of additional 300 mm waterline loops and connections are required in addition to the 600mm supply main running north-south into the IMF site. One loop will run west to provide water and fire flow capacity to the south half of Section 24 by tying in to the Stage I pipe located along the south limit of Section 24. The other waterline will be constructed along the east limit of the Loblaw's parcel to service the SE Section 19, NW Section 18 and Section 20 lands. The line will tie back into the 600 mm supply line at several locations as shown in Drawing 09 – Storm Water – Wastewater – Water – Phase I – Stage II.

Stage III - Long Term (beyond 25 years)

- For Stage III Phase I of Dry Light Industrial Development, the domestic flow capacity is increased by an additional 48.8 L/s i.e. maximum day demand for 192 Ha (256 Ha gross area).
- The upgrades for this level of development consist of some additional 300 mm waterline looping starting at the intersection of Dewdney Avenue and the IMF access running west for ½ mille along Dewdney Avenue, then south to intercept the 300 mm waterlines installed as part of Stage II. An additional 300 mm waterline will run south from Dewdney Avenue through the middle of Section 19. See Drawing 10 – Storm Water – Wastewater – Water – Phase I – Stage III.

4.3.5.2 Phase II

- In Phase II of Mixed Light Industrial Development the domestic maximum day flow demand is 96.6 L/s, i.e. maximum day demand for 273.8 Ha (365 Ha gross area).
- To aid in water circulation two 300 mm connections to the existing city system will be added at Read Avenue and at the GE Railcar and Pinkie Road waterlines; it will create a looped system that will reinforce service to the entire Industrial areas (1400 Ha). See Drawing 11 – Storm Water – Wastewater – Water – Phase II
- To supply water and fire flow capacity to the Mixed Industrial Developments in Section 21 and 28, a 300 mm waterline will be added to the system. It will run west from the intersection of Read Avenue and Toothill Street to Pinkie Road. It will then run south along Pinkie Road to intercept the 600 mm waterline on Dewdney Avenue. Another 300 mm connection will be made at GE Railcar near Pinkie Road to create a water system loop. Another waterline will run midway in Section 28, south to connect again to the 600 mm waterline on Dewdney Avenue.
- To supply water to Section 21 a connection will be made to the 600 mm trunk at Courtney Street and a 300 mm waterline will be installed in an east-west orientation roughly through the middle of Section 21 to Pinkie Road. From Pinkie Road it will run north to Dewdney Avenue. To reinforce the water system another waterline will run north midway through Section 21.

Figure 32 – Proposed Water Distribution Network for Entire IMF and Industrial Lands Development shows the proposed network for the entire Phase I and Phase II development.

Figure 32 – Proposed Water Distribution Network for Entire IMF and Industrial Lands Development

4.3.6 Water System Cost Estimates

Cost estimates for the proposed water distribution system servicing for the various phases and stages are summarized in Table 20 – Conceptual Water System Cost Estimate. The total cost of the first stages of development is expected to be approximately $17M. Total cost of the entire water servicing for the complete development is approximately $21M.

Table 20 – Conceptual Water System Cost Estimate

Phase	Stage	Description	Cost	Total
I	I	Water Pipes	11,100,000	
			Subtotal for Stage I	$11,100,000
I	II	Water Pipes	3,900,000	
			Subtotal for Stage II	$3,900,000
I	III	Water Pipes	1,200,000	
			Subtotal for Stage III	$1,200,000
			Total for Phase I	**$16,200,000**
II		Water Pipes	4,200,000	
			Total for Phase II	**$4,200,000**
			Total for Phase I & II	**$20,400,000**

4.3.7 Water Servicing Summary

Water servicing analysis was conducted for the proposed IMF and Industrial Lands development. The existing City of Regina WaterCAD model was converted into the EPANET model and servicing required to meet maximum day demands plus fire flows of 158 L/s (2500 USgpm) was established.The total water servicing costs for the first stages of development is approximately $17M. Most of the cost will be incurred by bringing a large diameter (600 mm) supply line into the IMF site from Lewvan Drive via 13th Avenue.

5. Conclusions and Recommendations

5.1 General

- The City is proposing to annex approximately five (5) sections of land within the Rural Municipality (RM) of Sherwood to plan for future industrial growth on the west side of the City as shown on Drawing 01. The annexation includes a proposed new IMF for CPR and a large warehouse facility for Loblaws Companies Ltd.
- In order to plan for the annexation of this land, a concept plan and municipal serviceability analysis was conducted. This report is the result of the serviceability analysis which included transportation, water, wastewater and storm services.
- A planning study conducted in parallel with this serviceability study produced a conceptual land use plan for the area as shown on Drawing 02. Land use is expected to be restricted to Dry Industrial and Mixed Industrial based on discussions with various stakeholders in the area.
- The planning study also provided a concept plan for the expected phasing and staging of the proposed IMF and Industrial Land development as shown in Drawing 03. Phase I is the proposed annexed area located west of Pinkie Road while Phase II is the proposed annexed area located east of Pinkie Road up to the west City Limits. Phase I has been split up into three (3) stages with the first stage, Stage I, expected to be developed within the next 0-3 years. The second and third stages are expected to be developed within the next 25 years and beyond 25 years, respectively. The remaining Phase II is expected to be developed beyond 25 years.

5.2 Transportation

- The goal of the transportation review is to confirm road network deficiencies and to identify any additional network upgrades required to support the fully developed IMF industrial growth area.
- Vehicle and truck trips are based on the number of employees forecast for each individual land use within the concept plan. Where available, actual employment estimates and/or trip information as provided by CPR and Loblaw's for their sites were used.
- A total of 13,695 employees are forecast for this area. The total number of trips generated based on the employment forecast is estimated at 5,685 trips (1,460 trips in and 4,225 trips out) during the weekday afternoon peak hour. Truck percentages are estimated at 15 percent, corresponding to a total of 855 trucks per hour during the afternoon peak hour. The average trip rate is 0.5 trips per employee. The distribution of trips is on average 25 percent inbound and 75 percent outbound during this time period.
- Total forecast afternoon peak hour traffic volumes were prepared for the 25 year forecast with development from Phases 1 and 2, and beyond 25-year forecast, which assumes full development of the study area. A capacity analysis was conducted on the forecast turning movement counts using Synchro and SimTraffic Version 7.0.

- Highway No. 1 and Pinkie Road was analyzed as an at-grade intersection with stop control for north-south movements. The largest movement is the southbound left turn (590 vph for the 25-year forecast and 1,310 vph for the long-term +25-year forecast). The southbound left turn movement will operate at a LOS F (i.e. delays of over 50 seconds per vehicle) and a volume to capacity (v/c) ratio of greater than 1.0 for both the 25-year forecast and the beyond 25-year forecast. The long delays may result in these drivers taking unacceptable risks in an attempt to enter the highway before it is safe to do so. An interchange at this location will be required prior to full development of Phases 1 and 2.
- The intersection of Dewdney Avenue and Pinkie Road was analyzed as an at-grade intersection with signalization. For the 25-year forecast, there will be ample capacity at this intersection to accommodate increase in traffic, assuming auxiliary right and left turn lanes are provided for each approach, as well as dual through lanes for each approach. With this geometry, the intersection will operate at a LOS B and an ICU of 59%, with all individual movements operating at a LOS C or better
- For the beyond 25-year forecast and full development of the industrial area, there will be limited capacity at the Dewdney Avenue and Pinkie Road intersection to accommodate all traffic, assuming the same geometry. It is recommended that in the long term (beyond 25 years), dual northbound and westbound left turn lanes be provided. This configuration will improve the intersection operations to a LOS C and an ICU of 77 percent. All individual movements will improve to a LOS D or better.
- A review of the available capacity and required cross section for Dewdney Avenue was conducted. The traffic volumes generated by the industrial area and assigned to Dewdney Avenue for the 25-year forecast will range from 1,600 vph east of Pinkie Road and 1,150 vph west of Pinkie Road, resulting in a LOS D and LOS B, respectively, for a 2-lane roadway. For the long-term (beyond 25-year) forecast, the traffic volumes will range from 3,100 vph east of Pinkie Road and 2,350 vph west of Pinkie Road, resulting in a LOS F, suggesting that a 4-lane cross section will be required in the long term.
- The only road network improvement included in the costing exercise is the upgrade of Dewdney Avenue from Lewvan Drive to the IMF access road. The total cost is estimated at $13.8 Million. The cost estimate assumes a strengthening of the existing 4-lane section from Lewvan Drive to Courtney Street, and the strengthening and widening to a new 4-lane cross section from Courtney Street to the IMF access road. The estimate also includes signals at the intersection of Pinkie Road and Dewdney Avenue and at the intersection of Courtney Street and Dewdney Avenue.

5.3 Municipal Servicing

5.3.1 Stormwater Servicing

- The proposed IMF and Industrial Lands development is situated on cultivated agricultural land. The general topography and flow patterns for drainage in the area are shown on Drawing 04. A divide starts at Section 21 with the lands east of the divide draining to Wascana Creek and the lands west draining to Cottonwood Creek via the Grand Coulee Drainage Ditch and tributaries.
- The XPSWMM modelling platform was used to estimate the predevelopment flow rates off of the existing site for a 1:100 year design storm in order to estimate the baseline conditions and to determine an appropriate future drainage system for the developed site. The future drainage system was designed to a 1:100 year design storm and configured assuming private dry bottom detention

facilities for the various parcels of land. These dry bottom detention facilities would be serviced with regional public open channel, as shown on Drawing 05.

- The predevelopment flow rates were modelled by adjusting the runoff factors such that they produced runoff indicative of rural development. For example, the area flowing towards Cottonwood Creek which has a catchment area of approximately 1197 Ha was modelled with an estimated peak flow of 5.55 m3/s or a flow rate of approximately 3.36 l/s/Ha (30 cfs/sq. mile). This flow is consistent with rural drainage in a flat prairie setting.
- The regional storm water channel required to accommodate the predevelopment 1 in 100 year rainfall event for the area draining to Cottonwood Creek has a conceptual design bottom width of 3.0 m and channel depths that range from 1.35 m to 2.0 m. The top width ranges from 16.5 m in the upstream reaches to 23.0 m at the downstream reaches from inside shoulder to inside shoulder. The noted widths do not include allowances for maintenance access, as suggested in the Development Standards Manual.
- The total Class 'D" cost estimate to construct the channel is $2,990,000.

5.3.2 Wastewater Servicing

- The only major wastewater infrastructure that currently exists in the vicinity of the proposed development is the Wastewater Treatment Plant (WWTP) and the McCarthy Booster Pumping Station (MBPS) Forcemains.
- Existing capacity of the WWTP is in the order of 250 MLD although it can temporarily accommodate close to 300 MLD for a short period of time.
- A convenient tie-in to the existing infrastructure is situated near the terminus of the MBPS forcemains which consists of a Valve Chamber (VC) located at the southeast corner of the WWTP site. This tie-in location will have a relatively consistent pressure boundary condition regardless of the hydraulic loads produced in other parts of the wastewater system and therefore it was chosen as the tie-in for the proposed wastewater servicing concepts.
- Wastewater sewer flows for various phases and stages of development were derived using design parameters consistent with the City of Regina Development Standards. The base flows were estimated using a per capita average flow demand of 225 lpcd and the Harmon's Peaking Factor (HPF) equation. Equivalent populations required for the HPF equation were estimated based on 25 people/Ha for Dry Industrial land use and 35 people/Ha for Mixed Industrial land use. Groundwater induced flow (GWI) and stormwater induced flow (SWI) were estimated using a static factor of 31,100 l/Ha/day. This factor is equivalent to induced extraneous flows from a 25 year design storm event with weeping tile flows connected to the wastewater system.
- The total estimated peak wastewater flow for Phase I Stage I is 10.5 MLD, 34.4 MLD for Phase I Stage II and 45.8 MLD for Phase I Stage III, i.e. complete Phase I buildout. For Phase II the total estimated flow is 23.6 MLD. The total flow from both Phase I and Phase II combined is 69.4 MLD which represents roughly 28% of the current WWTP capacity of 250 MLD.
- Wastewater servicing for Phase I and Phase II was developed using the estimated peak wastewater flows based on the parameters described above and design spreadsheets included in Appendix B. The servicing was configured as two separate systems, each with its own pump station and forcemain as shown on Drawing 06.

- The expected wastewater system response to the proposed IMF and Industrial lands development was modeled using the InfoWorks CS modeling platform from Wallingford Software. The InfoWorks CS modeling platform is a dynamic hydraulic simulation software package capable of modeling complex flow hydraulics and loading conditions such as the diurnal flow patterns associated with different land use types.
- The model included portions of the existing wastewater interceptor system developed from previous servicing and operational studies conducted by UMA Engineering, including the SW Sector and NW Sector servicing studies and the MBPS forcemain study.
- The primary focus for the system response analysis was to review the impacts of the proposed development on the SW Sector of Regina which is the area most susceptible to basement flooding during large rainstorm events. Both the South Interceptor Trunk and Wascana Interceptor Trunk were reviewed under various loading conditions resulting from the proposed IMF and Industrial Lands development.
- The Phase I Stage I development which includes the proposed IMF and the Loblaws warehouse had an estimated static peak hydraulic loading of 10.5 MLD. Under dynamic loading conditions this flow was reduced to roughly 8 MLD. Total loading at the WWTP was increased from 287 MLD under existing conditions with a 25 year design rainstorm event to 295 MLD which is very close to the absolute maximum flow rate that the WWTP can accommodate at this time. The affect on the hydraulic grade lines in the South and Wascana Interceptor Trunks was minimal. Therefore, the proposed development could likely proceed without requiring additional improvements to the remaining wastewater regional infrastructure.
- The existing wastewater system was also checked with the entire IMF and Industrial Lands development added to the system, i.e. Phase I Stage III and Phase II added. Under these conditions, the flows to the WWTP under static conditions was estimated to be 69.4 MLD using the design parameters described previously. Under dynamic flow conditions the hydraulic loads from the proposed development are reduced to roughly 50 MLD, however, this condition produces flows at the WWTP that are significantly higher than the maximum plant capacity. Flows to the WWTP with the entire Phase I and Phase II added is estimated to be 337 MLD versus the existing condition of 287 MLD under a 25 year design storm event.
- The effects on the South Trunk and Wascana Trunk are also quite significant with Phase I and Phase II added. The increase in the hydraulic grade lines on these two trunks in the southwest quadrant of the City would likely increase the occurrences of basement flooding in these areas and some sort of relief in the area would likely be necessary.
- The final analysis that was conducted on the wastewater system was the expected system response under future loading conditions with Phase I and Phase II added, i.e., the expected response for a wastewater system configured to accommodate a future target population of 235,000 people. This analysis was accomplished by taking the future Stage I and Stage 2 loading conditions presented in the SW Servicing Study assuming in-line storage as the preferred servicing option and the future Stage 1 loading conditions from the NW Sector Servicing Study. The other modification to the existing wastewater system was twinning of the South Trunk. Under these conditions, it was determined that the South Trunk twinning would not provide enough relief to overcome the additional impacts that result from the IMF and Industrial Lands Development. Therefore, additional upgrading to the existing wastewater system would likely be required under these conditions. The most obvious

upgrading would be at the WWTP site where the existing 1350 mm pipe leading from the VC to the Primary Treatment Plant presents a significant hydraulic constraint and the capacity at this location should be upgraded along with the WWTP capacity.

5.3.3 Water Servicing

- Similar to the wastewater servicing, very little public water infrastructure exists in the vicinity of the proposed development and the infrastructure that does exists was found to have limited capacity. The proposed development cannot be accommodated without significant improvements to the regional water distribution system.
- Two servicing options were reviewed, however, the only one that proved feasible was a 600 mm trunk connected to the City Loop supply system. The closest tie-in location was at Lewvan Drive and 13th Avenue.
- The water servicing analysis was conducted using the EPANET software package and the City of Regina's existing water distribution system model that was developed using the WaterCAD platform from Haestad Methods. The WaterCAD software uses EPANET as its computational engine so the results from the two packages will be identical.
- Water demands from the proposed development were estimated using the parameters included in the City of Regina's Development Standards. Average Day Consumption was set to 415 lpcd. Maximum Day and Maximum Hour Peaking Factors were set to 2.1 and 3.2 times average day, respectively. Equivalent populations for Dry Industrial and Mixed Industrial land uses were estimated using population densities of 25 people/Ha and 35 people/Ha, respectively. The minimum fire flow criteria were set to 158 L/s (2500 USgpm) and a minimum pressure in the surrounding water distribution network of 140 kPa.
- In order to simulate the expected future hydraulic conditions, the demands in the existing water distribution system were increased by adjusting the demand multipliers so that total maximum day demand from the two existing reservoirs produced a total flow of 205 MLD. Under maximum hour conditions, the demand multipliers were adjusted so that the flow rate from the two reservoirs was 312 MLD. These values correspond to a total population of 235,000 people using an average day demand of 415 lpcd and peaking factors for maximum day and maximum hour of 2.1 and 3.2, respectively.
- The proposed water distribution system servicing for the entire development using the assumptions described above is shown on Drawing 07. Note that the 600 mm feed to Lewvan Drive is not shown in its entirety. The entire proposed water distribution network servicing the entire IMF and Industrial Lands development can be seen in Figure 32 – Proposed Water Distribution Network for Entire IMF and Industrial Lands Development.

5.3.4 Phasing and Staging of Municipal Services and Cost Estimates

- Drawings 08 to 11 show the expected municipal servicing requirements for the various Phases and Stages of the IMF. Significant upfront servicing is required due to the remote nature of the initial areas of development and the complete lack of existing municipal infrastructure in the area.
- Table 21 – Conceptual Municipal Serviceability Cost Estimate below is a summary of the expected servicing costs for the various phases and stages of development based on the information shown on

Drawings 08 to 11. Note from the table that Phase I Stage I will cost roughly $37M for stormwater, wastewater and water servicing (excluding transportation). The total cost of the municipal servicing (excluding transportation) for Phases I & II is estimated to be roughly $88.5 M.

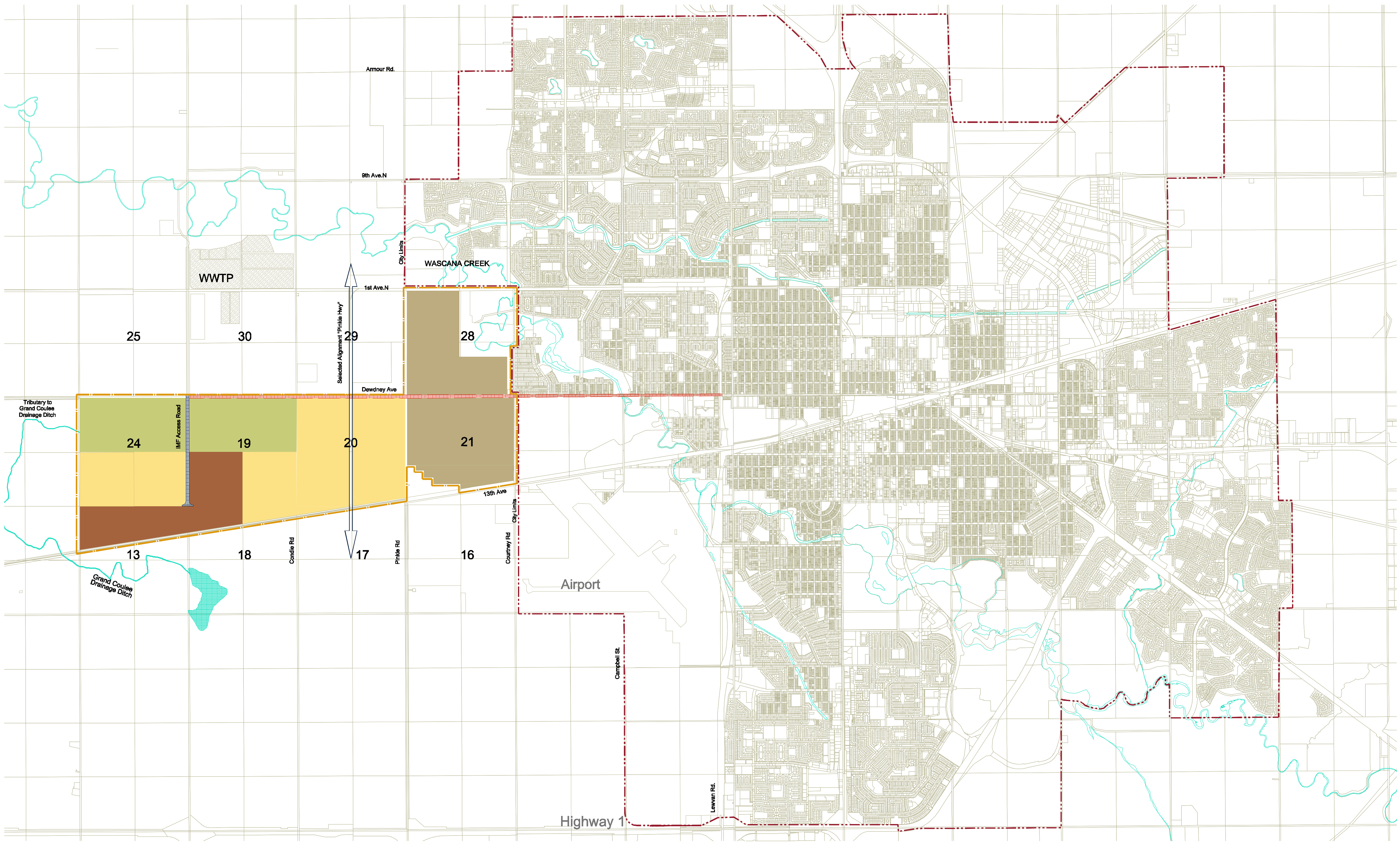
Table 21 – Conceptual Municipal Serviceability Cost Estimate

Phase	Stage	Description	Cost	Total
I	I	Storm Water System	1,150,000	
		Wastewater System	24,610,000	
		Water System	11,100,000	
			Total for Stage I	$36,860,000
I	II	Storm Water System	1,430,000	
		Wastewater System	17,000,000	
		Water System	3,900,000	
			Total for Stage II	$22,330,000
I	III	Storm Water System	410,000	
		Wastewater System	4,000,000	
		Water System	1,200,000	
			Total for Stage III	$5,610,000
			Total for Phase I	**$64,800,000**
II		Storm Water System	0	
		Wastewater System	19,350,000	
		Water System	4,200,000	
			Total for Phase II	**$23,550,000**
			Total for Phase I & II	**$88,350,000**

Appendix A

Report Drawings

List of Drawings



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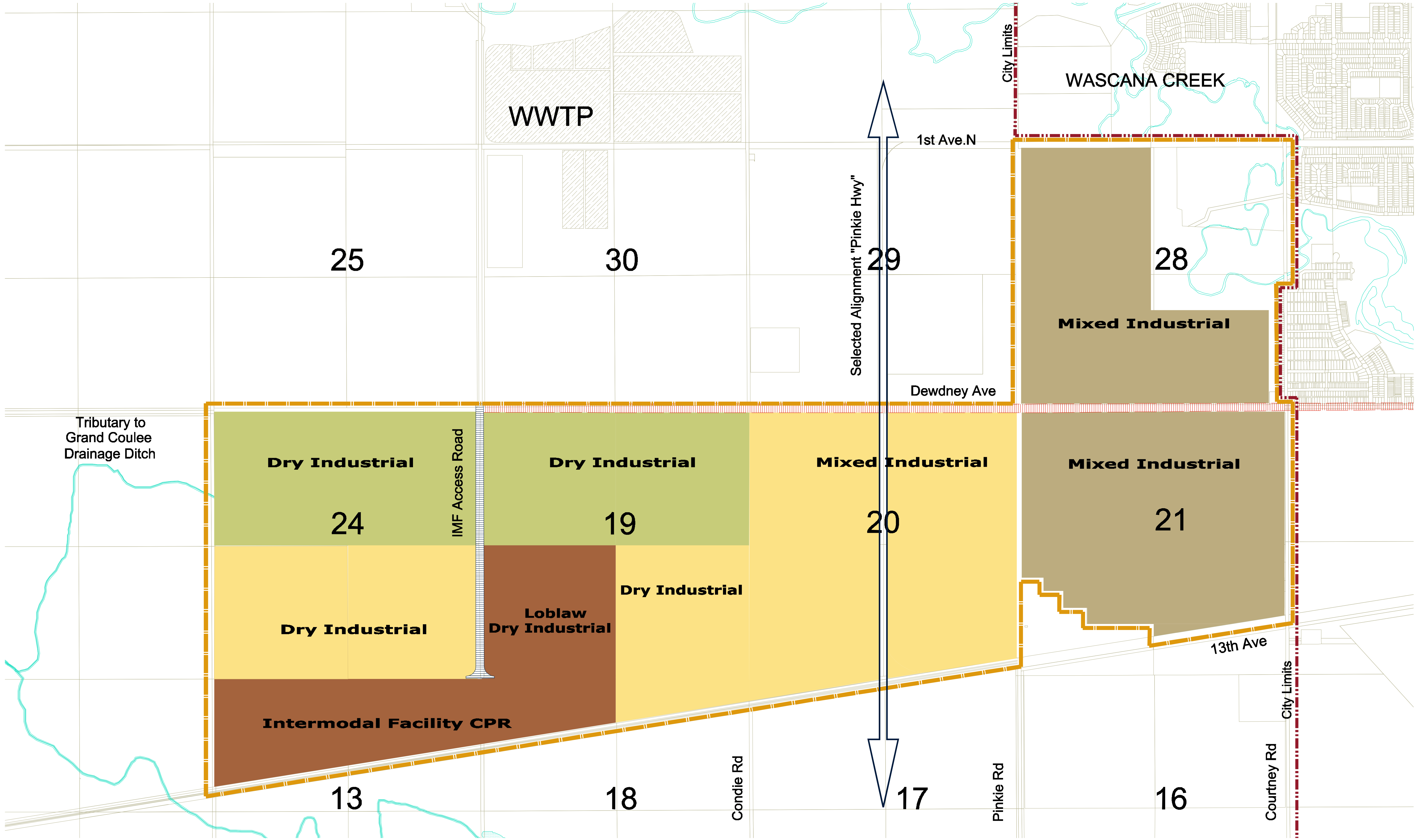
City of Regina

Intermodal Facility and Industrial Lands Servicing Study

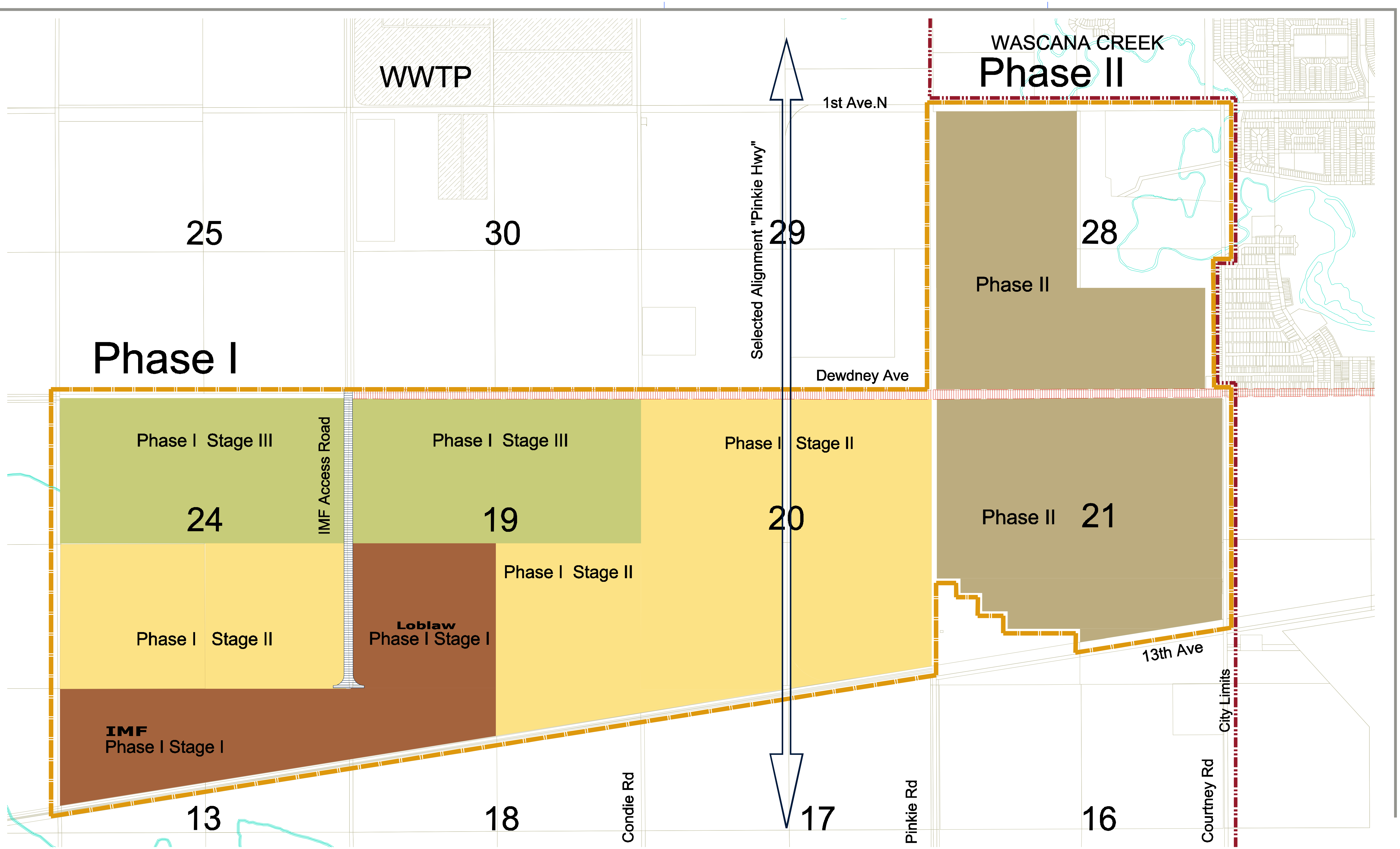
Annexation Areas Servicing Review

Drawing 01



City of Regina
Intermodal Facility and Industrial Lands Servicing Study
Land Uses - Servicing Review
Drawing 02

UMA | AECOM



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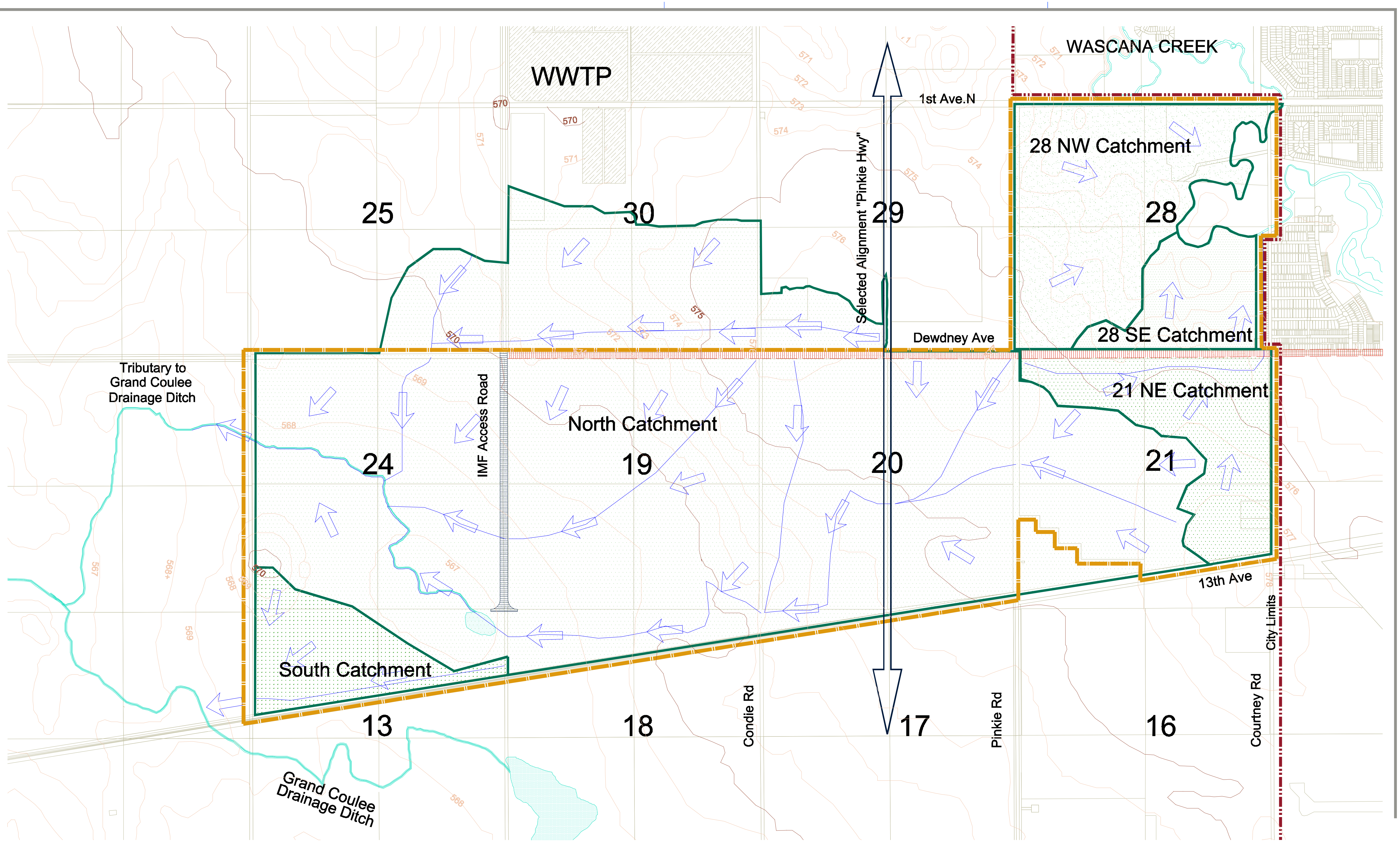
City of Regina
Intermodal Facility and Industrial Lands Servicing Study
Phasing and Staging - Servicing Review
Drawing 03



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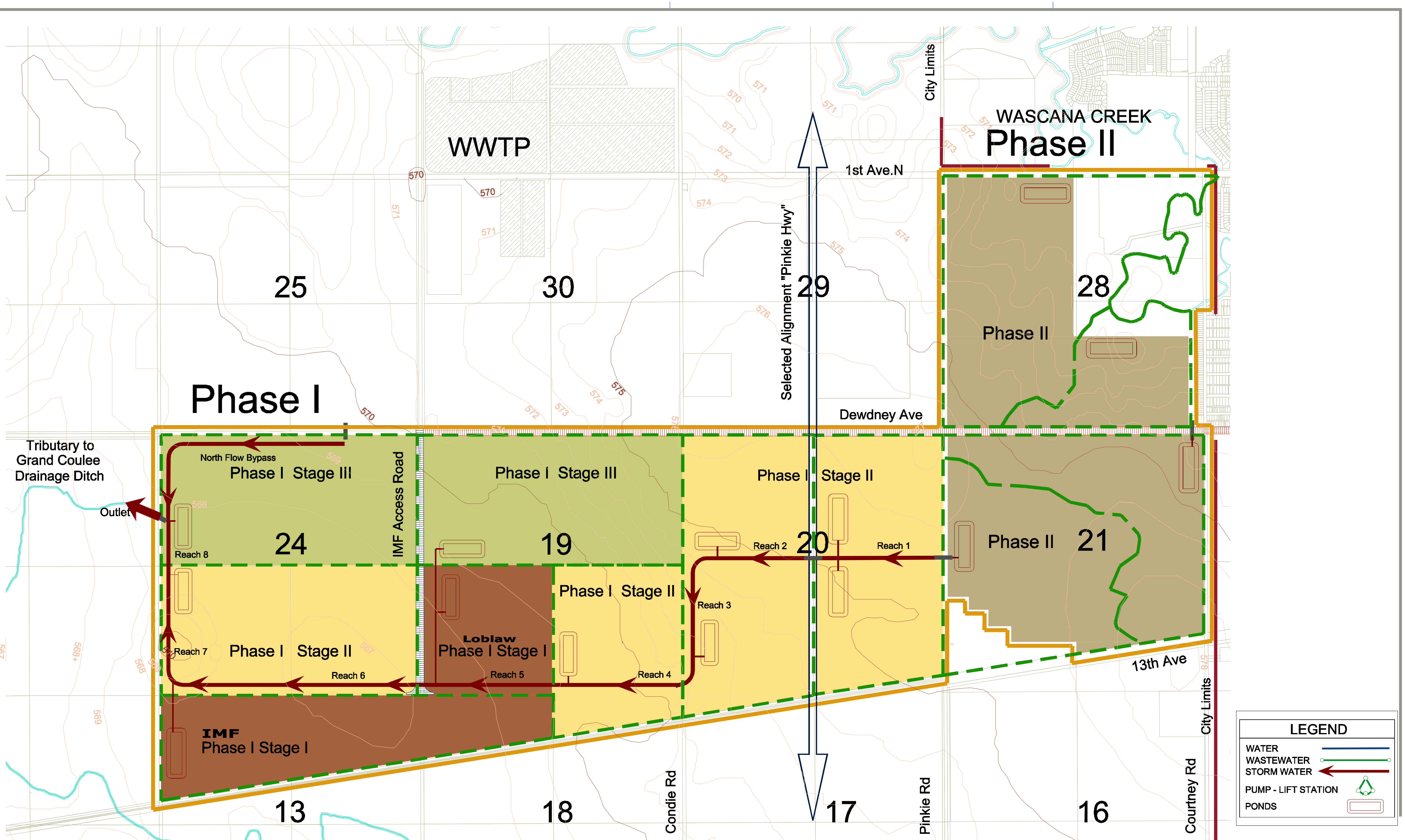
City of Regina

Intermodal Facility and Industrial Lands Servicing Study

Predevelopment - Storm Water Review

Drawing 04



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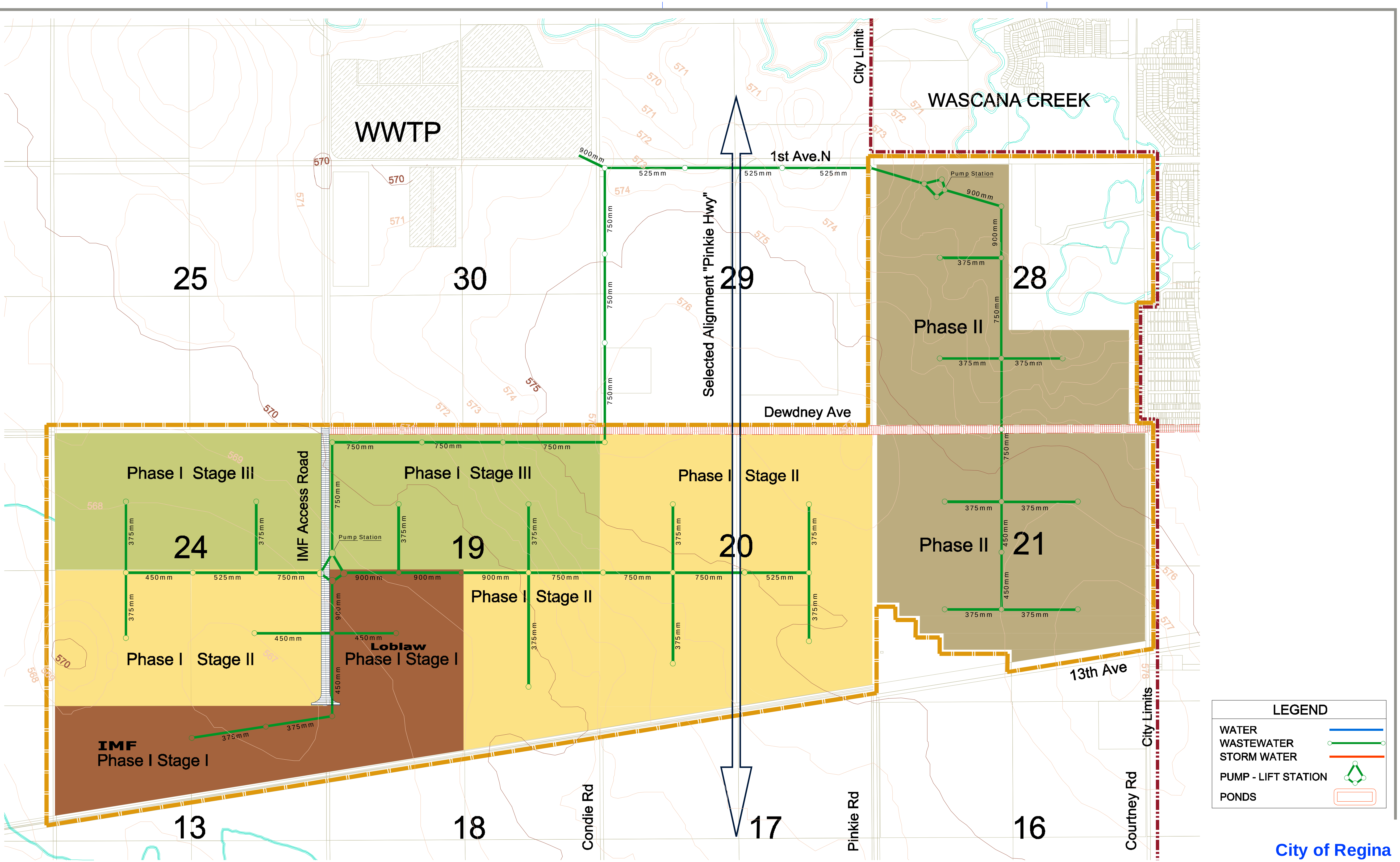
City of Regina
Intermodal Facility and Industrial Lands Servicing Study
Post Development - Storm Water Review
Drawing 05



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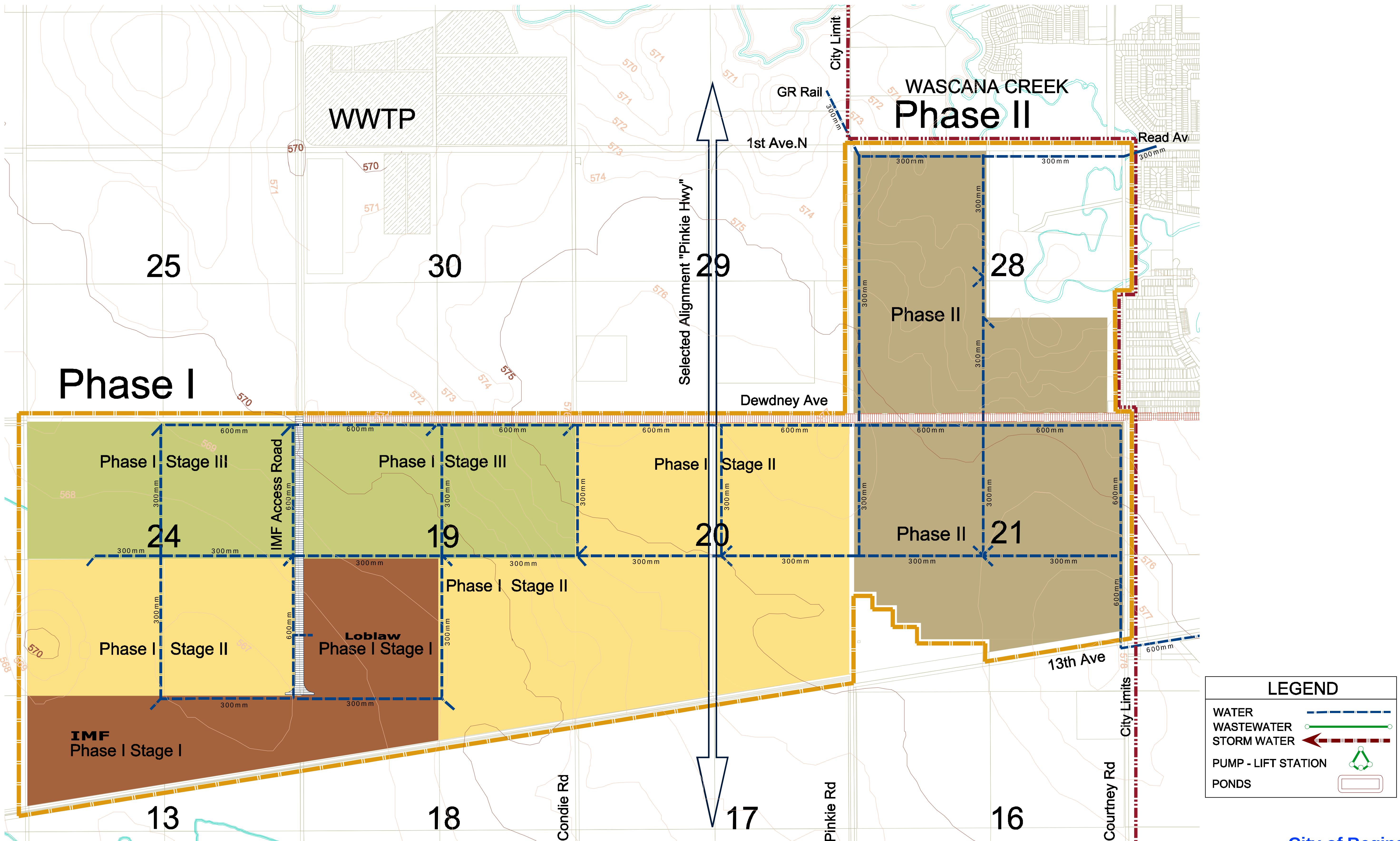
City of Regina

Intermodal Facility and Industrial Lands Servicing Study

Wastewater System - Servicing Review

Drawing 06



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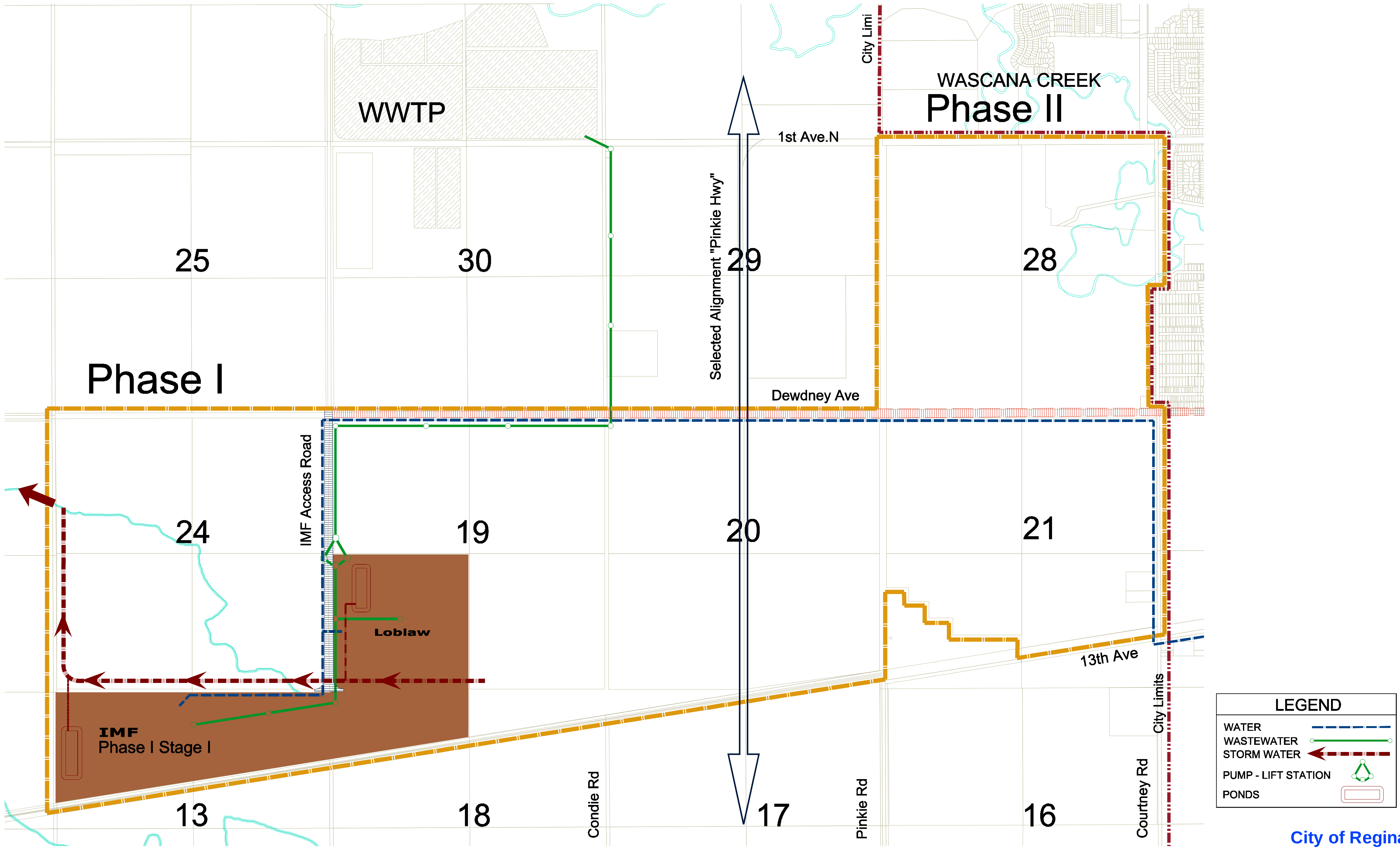
City of Regina
Intermodal Facility and Industrial Lands Servicing Study
Water System - Servicing Review
Drawing 07



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City of Regina
Intermodal Facility and Industrial Lands Servicing Study
Storm Water-Wastewater-Water Phase I - Stage I
Drawing 08

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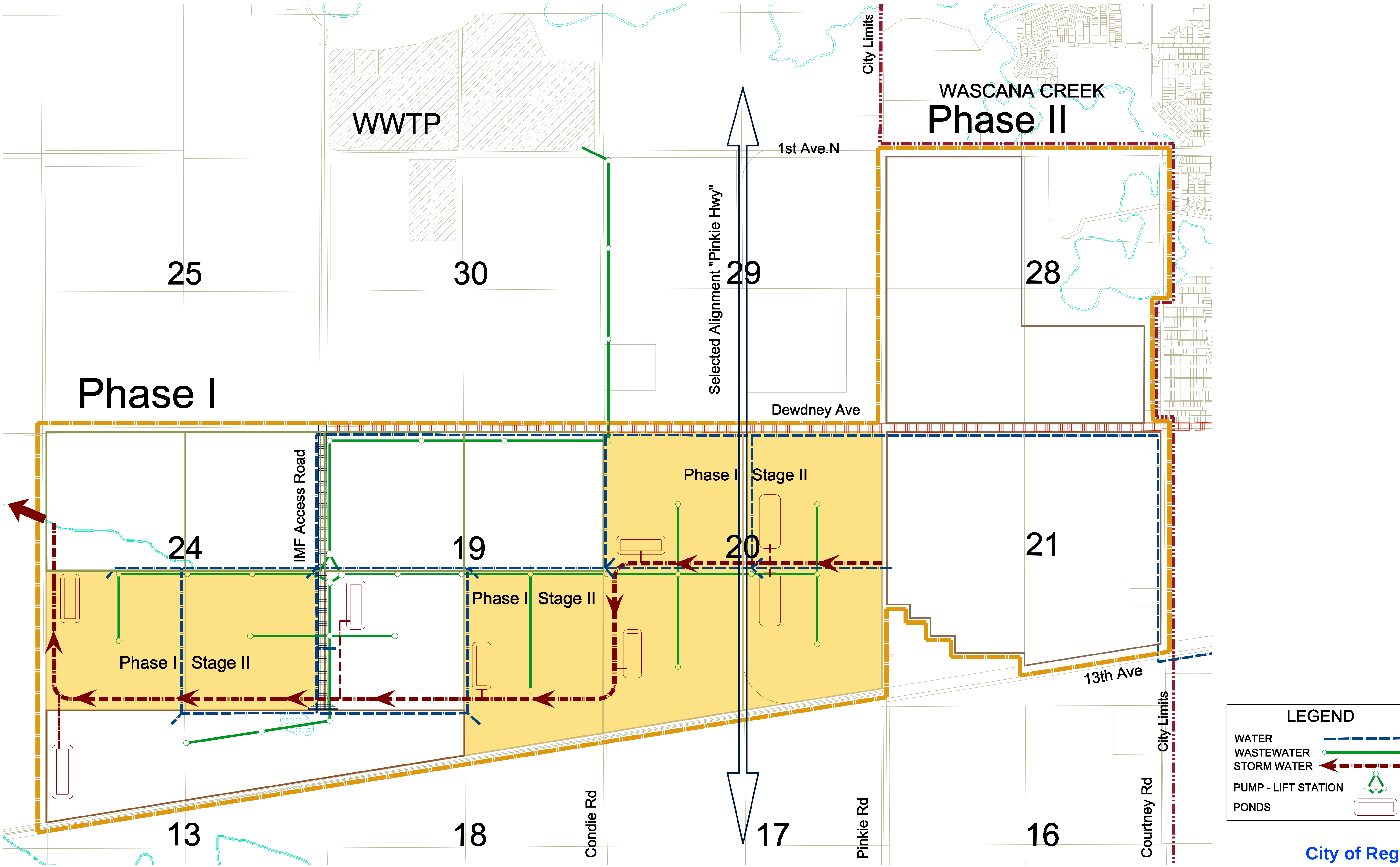

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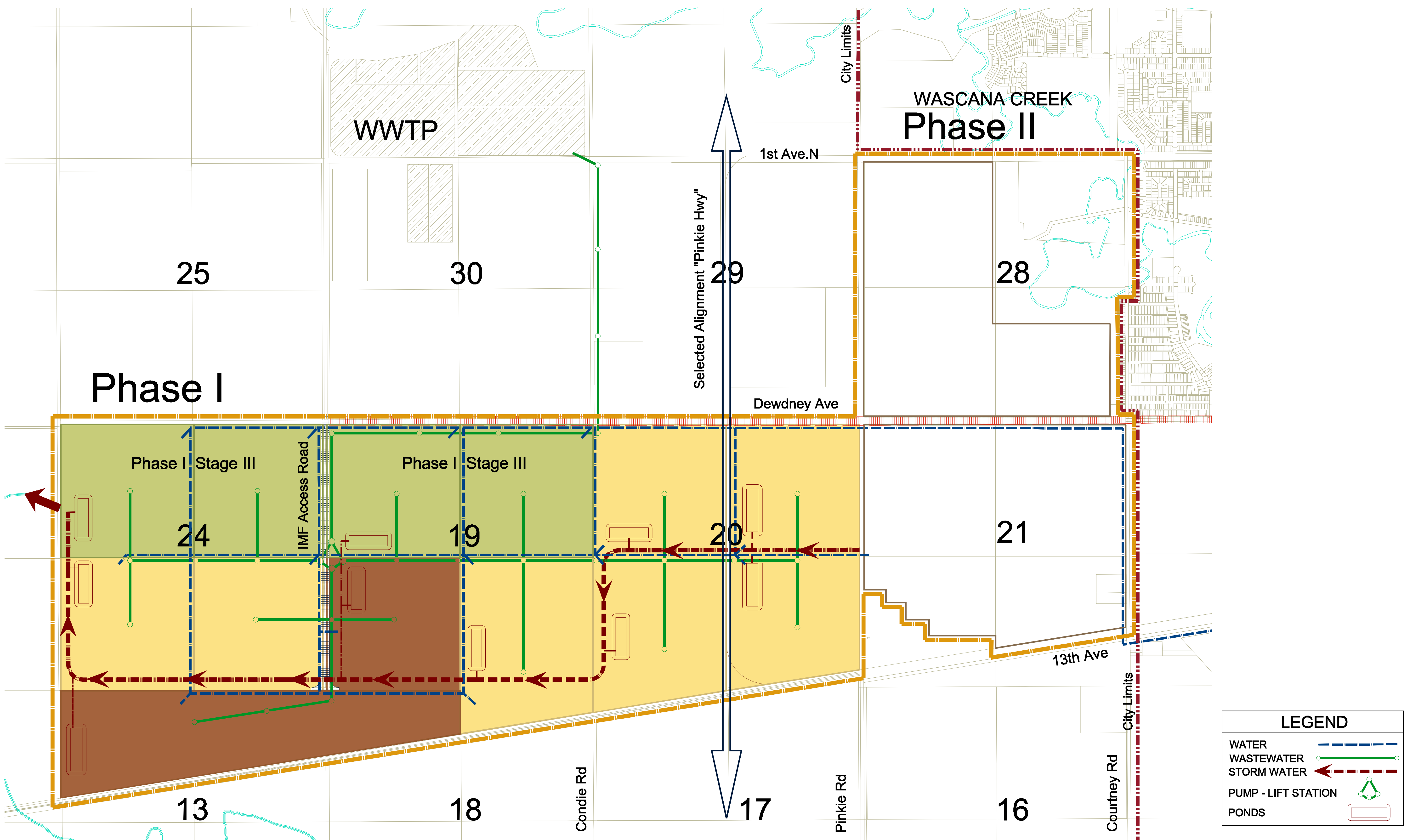
City of Regina

Intermodal Facility and Industrial Lands Servicing Study

Storm Water-Wastewater-Water Phase I - Stage II

Drawing 09



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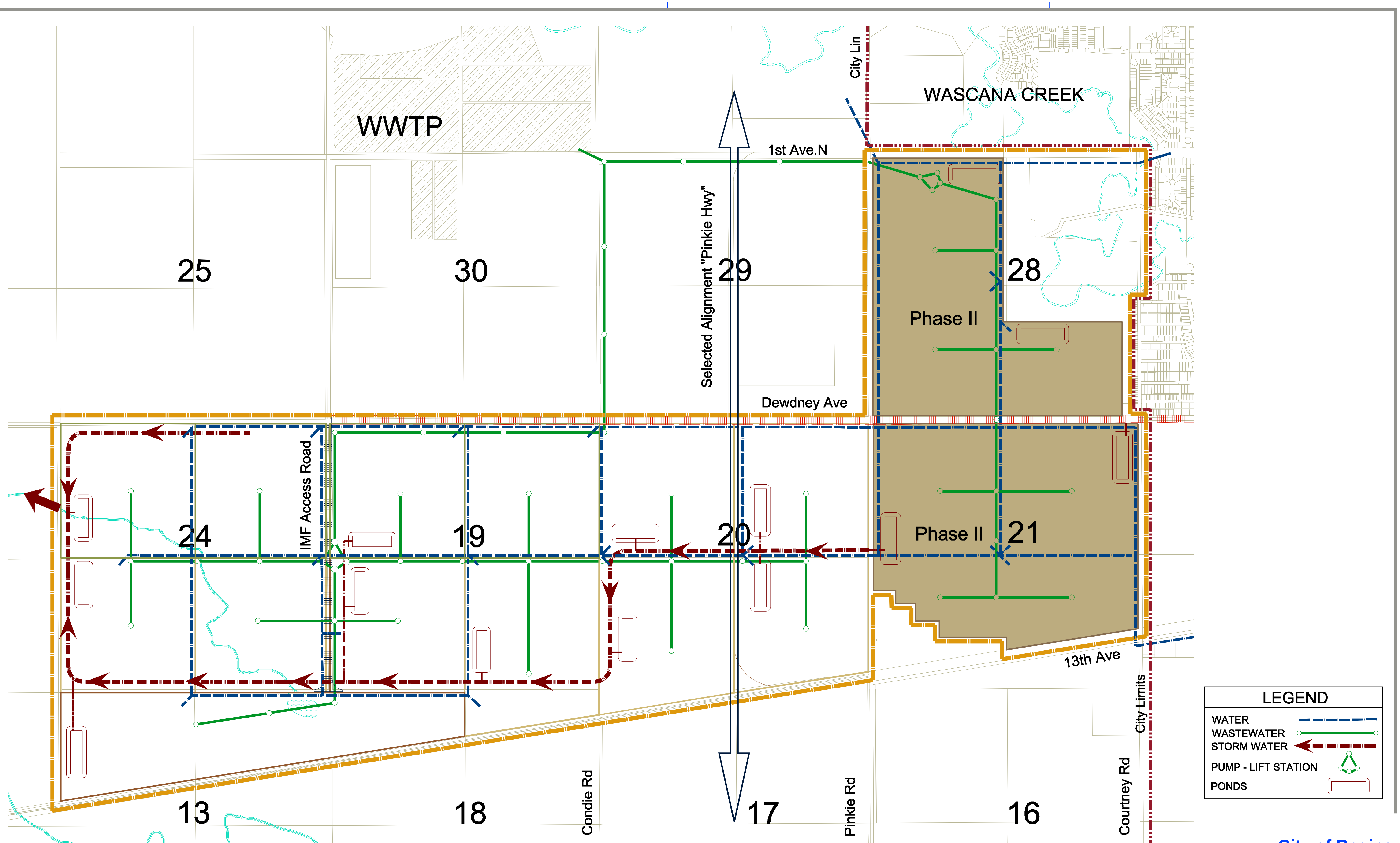
City of Regina

Intermodal Facility and Industrial Lands Servicing Study

Storm Water-Wastewater-Water Phase I - Stage III

Drawing 10



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City of Regina

Intermodal Facility and Industrial Lands Servicing Study

Storm Water-Wastewater-Water Phase II

Drawing 11

Appendix B

Wastewater Design Spreadsheets

WWS Servicing East Section - Wet Industrial Parameters
D. ENNS
21-Aug-08

IMF SERVICING AREA

Manhole depth - high end	
Drop at Manhole	

Mannings	0.013		
Sewage/dwelling unit	225	lpcd	
I&I Allowance	31100	l/Ha/day	

	From	To	Population	Total Population				Harmon Factor	Domestic Flow (l/sec)	Area (Ha)	Total Area (Ha)	I&I Flow (l/sec)	Total I&I Flow (l/sec)	Total Extraneous Flow (l/sec)	Institutional Flow (l/sec)	Total Wet Weather Flow (l/sec)	Length	Diameter (mm)	Slope	Full Capacity (l/sec)	% Full Flow	High End	US Ground	DS Ground	US Invert	DS Invert	Match Invert Elevation
	Sec21_SW	Sec21_S	1918	1918				3.60	17.98	54.8	54.8	19.73	19.73	19.73	0.00	37.71		375	0.140%	65.60	57.5%						
	Sec21_SE	Sec21_S	1657	1657				3.65	15.74	47.35	47.35	17.04	17.04	17.04	0.00	32.78		375	0.140%	65.60	50.0%						
	Sec21_S	Sec21_N	0	3575				3.38	31.44	0	102.15	0.00	36.77	36.77	0.00	68.21		450	0.110%	94.56	72.1%						
	Sec21_NW	Sec21_N	2263	2263				3.54	20.88	64.65	64.65	23.27	23.27	23.27	0.00	44.15		375	0.140%	65.60	67.3%						
	Sec21_NE	Sec21_N	2165	2165				3.56	20.06	61.85	61.85	22.26	22.26	22.26	0.00	42.33		375	0.140%	65.60	64.5%						
	Sec21_N	Sec28_S	0	8003				3.05	63.57	0	228.65	0.00	82.30	82.30	0.00	145.87		750	0.057%	265.79	54.9%						
	Sec28_SW	Sec28_S	2443	2443				3.52	22.37	69.8	69.8	25.12	25.12	25.12	0.00	47.50		375	0.140%	65.60	72.4%						
	Sec28_SE	Sec28_S	2098	2098				3.57	19.50	59.94	59.94	21.58	21.58	21.58	0.00	41.08		375	0.140%	65.60	62.6%						
	Sec28_S	Sec28_N	0	12544				2.86	93.31	0	358.39	0.00	129.00	129.00	0.00	222.31		750	0.057%	265.79	83.6%						
To Forcemain	Sec28_N	Pinkie.WW	3105	15649				2.76	112.47	88.7	447.09	31.93	160.93	160.93	0.00	273.40		900	0.045%	384.02	71.2%						

*Full Capacity calculated using Mannings equation: $Q = Cm/n*A*R^{2/3}*S^{1/2}$ Cm = 1.0 (S.I)

15649

447.09

IMF SERVICING AREA - INTERMODAL FACILITY AND SU
D. ENNS
21-Aug-08

IMF SERVICING AREA

(green)	Ph 1 Stg 1
(yellow)	Ph 1 Stg 2
(red)	Ph 1 Stg 3

Manhole depth - high end	
Drop at Manhole	

Mannings	0.013		
Sewage/dwelling unit	225	lpcd	
I&I Allowance	31100	l/Ha/day	

	From	To	Population	Total Population	Weeping Tile Residences	Harmon Factor	Domestic Flow (l/sec)	Area (Ha)	Total Area (Ha)	I&I Flow (l/sec)	Total I&I Flow (l/sec)	Total Extraneous Flow (l/sec)	Institutional Flow (l/sec)	Total Institutional Flow (l/sec)	Total Wet Weather Flow (l/sec)
Intermodal_W	Intermodal_W	Intermodal_E	1809	1809	0	3.62	17.05	72.35	72.35	26.04	26.04	26.04	0	0	43.09
Intermodal_E	Intermodal_E	IMF-Access.2	1002	2811	0	3.47	25.37	40.08	112.43	14.43	40.47	40.47	0	0	65.84
Sec19_SW	Sec19_SW	IMF-Access.2	1758	1758	0	3.63	16.61	70.3	70.3	25.30	25.30	25.30	16	16	57.92
Sec24_SE	Sec24_SE	IMF-Access.2	1785	1785	0	3.62	16.84	71.38	71.38	25.69	25.69	25.69	16	16	58.54
IMF Road	IMF-Access.2	IMF-Access.1	0	6354	0	3.15	52.07	0	254.11	0.00	91.47	91.47	0	32	175.54
Sec24_NW	Sec24_NW	Sec24_W	1528	1528	0	3.67	14.62	61.11	61.11	22.00	22.00	22.00	0	0	36.62
Sec24_SW	Sec24_SW	Sec24_W	1582	1582	0	3.66	15.09	63.26	63.26	22.77	22.77	22.77	0	0	37.86
	Sec24_W	Sec24_E	0	3110	0	3.43	27.77	0	124.37	0.00	44.77	44.77	0	0	72.54
	Sec24_NE	Sec24_E	1748	1748	0	3.63	16.53	69.91	69.91	25.16	25.16	25.16	0	0	41.69
	Sec24_E	IMF-Access.1	0	4858	0	3.26	41.20	0	194.28	0.00	69.93	69.93	0	0	111.13
Sec20_NE	Sec20_NE	Sec20_E	2271	2271	0	3.54	20.95	64.89	64.89	23.36	23.36	23.36	0	0	44.31
Sec20_SE	Sec20_SE	Sec20_E	2095	2095	0	3.57	19.48	59.85	59.85	21.54	21.54	21.54	0	0	41.02
Sec20_E	Sec20_E	Sec20_W	0	4366	0	3.30	37.51	0	124.74	0.00	44.90	44.90	0	0	82.41
	Sec20_NW	Sec20_W	1687	1687	0	3.64	16.00	67.46	67.46	24.28	24.28	24.28	0	0	40.28
	Sec20_SW	Sec20_W	1879	1879	0	3.61	17.65	75.14	75.14	27.05	27.05	27.05	0	0	44.70
	Sec20_W	Sec19_E	0	7932	0	3.05	63.08	0	267.34	0.00	96.23	96.23	0	0	159.31
	Sec19_NE	Sec19_E	1624	1624	0	3.65	15.45	64.97	64.97	23.39	23.39	23.39	0	0	38.84
	Sec19_SE	Sec19_E	2039	2039	0	3.58	19.01	81.57	81.57	29.36	29.36	29.36	0	0	48.37
	Sec19_E	Sec19_W	0	11595	0	2.89	87.28	0	413.88	0.00	148.98	148.98	0	0	236.26
	Sec19_NW	Sec19_W	1683	1683	0	3.64	15.97	67.31	67.31	24.23	24.23	24.23	0	0	40.19
	Sec19_W	IMF-Access.1	0	13278	0	2.83	97.91	0	481.19	0.00	173.21	173.21	0	0	271.11
To Forcemain	IMF-Access.1	IMF_WW	0	24490	0	2.56	163.55	0	929.58	0.00	334.61	334.61	0	32	530.16

From	To	Length	Diameter (mm)	Slope	Full Capacity (l/sec)	% Full Flow	High End	US Ground	DS Ground	US Invert	DS Invert	Match Invert Elevation
Intermodal_W	Intermodal_E		375	0.140%	65.60	65.7%						
Intermodal_E	IMF-Access.2		450	0.110%	94.56	69.6%						
Sec19_SW	IMF-Access.2		450	0.110%	94.56	61.2%						
Sec24_SE	IMF-Access.2		450	0.110%	94.56	61.9%						
IMF-Access.2	IMF-Access.1		750	0.057%	265.79	66.0%						
Sec24_NW	Sec24_W		375	0.140%	65.60	55.8%						
Sec24_SW	Sec24_W		375	0.140%	65.60	57.7%						
Sec24_W	Sec24_E		450	0.110%	94.56	76.7%						
Sec24_NE	Sec24_E		375	0.140%	65.60	63.6%						
Sec24_E	IMF-Access.1		600	0.077%	170.38	65.2%						
Sec20_NE	Sec20_E		375	0.140%	65.60	67.5%						
Sec20_SE	Sec20_E		375	0.140%	65.60	62.5%						
Sec20_E	Sec20_W		525	0.092%	130.44	63.2%						
Sec20_NW	Sec20_W		375	0.140%	65.60	61.4%						
Sec20_SW	Sec20_W		375	0.140%	65.60	68.1%						
Sec20_W	Sec19_E		750	0.057%	265.79	59.9%						
Sec19_NE	Sec19_E		375	0.140%	65.60	59.2%						
Sec19_SE	Sec19_E		375	0.140%	65.60	73.7%						
Sec19_E	Sec19_W		900	0.045%	384.02	61.5%						
Sec19_NW	Sec19_W		375	0.140%	65.60	61.3%						
Sec19_W	IMF-Access.1		900	0.045%	384.02	70.6%						
IMF-Access.1	IMF_WW		1200	0.031%	686.44	77.2%						

*Full Capacity calculated using Mannings equation: $Q = Cm/n*A*R^{2/3}*S^{1/2}$ Cm = 1.0 (S.I)

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